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DISCUSSIONS

APPLICATIONS FOR ADMISSION AND TRANSFER

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SOIL MECHANICS RESEARCH[1]

By Glennon Gilboy,[2] Jun. Am. Soc. C. E.

Synopsis

In the fall of 1925, Charles Terzaghi, M. Am. Soc. C. E., joined the staff of Massachusetts Institute of Technology for the purpose of inaugurating advanced courses and establishing a program of extensive research in the field of Soil Mechanics. Under his direction a great amount of pioneer work was carried on, which served firmly to establish the importance of research in foundation engineering and allied subjects, and to define clearly the most promising lines of endeavor.

It is the purpose of this paper to summarize briefly the salient elements of this work. The problems described fall into two main groups: (1) Soil Physics, comprising studies of the composition and grain distribution of soils, permeability, compressibility, consolidation, compressive strength, internal friction, and cohesion; and (2) Soil Engineering, including investigations on the bearing capacity and settlement of foundations, on hydraulic-fill dams, on the frost heaving of highway sub-grades, and on the lateral pressure of earth against retaining walls.

The first group includes studies of the physical properties of soils and of the interrelation between these properties, with the ultimate object of understanding the nature of the effects observed in soils under various conditions. The second group comprises investigations of the behavior of soil as an engineering material forming part of a structure. The two groups are closely interrelated, inasmuch as a knowledge of the behavior of a soil mass connotes a knowledge of the physical properties of its component parts.

In the last analysis, engineering problems control the trend of development of soil mechanics. A thorough knowledge of soil physics in itself is interesting and valuable from a scientific standpoint; but it is economically worthless unless it provides a means for answering the questions that puzzle

[1] Presented at the meeting of the Structural Division, Boston, Mass., October 11, 1929. Written discussion on this paper will be closed in January, 1932, Proceedings.

[2] Asst. Prof. of Soil Mechanics, Mass. Inst. Tech., Cambridge, Mass.

the practicing engineer. It has been the aim of Dr. Terzaghi and his associates to keep this principle constantly in mind, and to plan the research work accordingly.

Research in Soil Physics

Mechanical Analysis.—The oldest and most obvious method of soil analysis is the separation of a soil sample into fractions on the basis of grain size. The results of such an analysis are usually represented by a curve showing the weight of all grains smaller than any given diameter, expressed as a percentage of the total weight of the sample. A typical curve is shown in Fig. 1. Point *A*, for example, indicates that 30% of the sample is made up of grains smaller than 0.1 mm. in diameter.

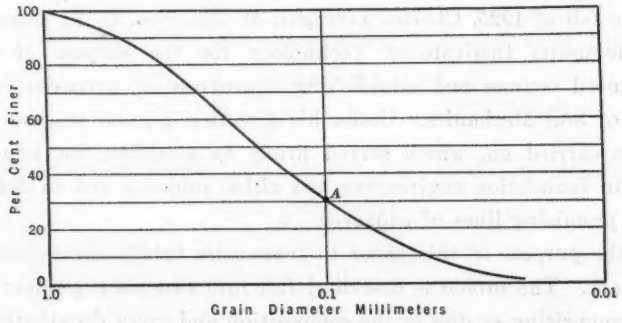

Fig. 1.—Typical Mechanical Analysis Diagram

For coarser materials, such as sand and gravel, the analysis is made in a simple manner by passing the sample through a series of screens. In separating soils composed of grains which are finer than the openings in the finest screen available, one of the many sedimentation methods may be used. Such methods are based on the principle that the rate at which a sphere sinks in a liquid is directly proportional to the square of the diameter of the sphere.

Considerable research on sedimentation has been done at the Massachusetts Institute of Technology with a view to standardizing equipment and procedure. Simple sedimentation in jars was first used, but it was found tedious and inaccurate. Attention was next directed to Wiegner's method.[3] In this system the soil suspension is contained in a long vertical cylinder, which communicates near its lower end with a small stand-pipe. The latter contains distilled water. Since the specific gravity of the suspension is greater than that of water, the level in the stand-pipe is higher than that in the cylinder. As sedimentation proceeds, the difference in level decreases, approaching zero as the suspension clears. A plot of the difference in level against time affords a means of constructing graphically a mechanical analysis curve for the sample.

[3] "Uber eine neue Methode der Schlämmanalyse," von G. Wiegner, *Zentralblatt* für die gesamte Landwirtschaft, Bd. I (1920), No. 1.

The principal source of error in the simplest form of the apparatus is the difficulty of making an accurate visual measurement of the level difference, which is initially about 1 cm. Gessner has devised a photographic apparatus which is reported to give good results.[4] It was felt, however, that something simpler and less expensive would be advisable. The scheme finally adopted was based on magnification of the level difference by means of an inclined gauge glass connected to the stand-pipe. After the apparatus had been carefully calibrated and charts had been worked out to simplify the computations, the method was used successfully for a considerable length of time.

An even simpler method was later developed from ideas suggested by the investigations of Professor Bouyoucos.[5] The basic principle resembles Wiegner's, in that it involves time observations on the variation in density of a soil suspension as the particles settle. Instead of a piezometer, however, an immersion hydrometer is used to measure the density. In co-operation with the United States Bureau of Public Roads extended investigations have been carried out in order to develop in full the underlying theory of the method and to devise testing procedures which would furnish accurate results. A special hydrometer was designed to minimize errors, and charts were constructed so that computations involving corrections for temperature, density, and viscosity of the medium, specific gravity of the solid particles, and the flotation characteristics of the hydrometer, might be performed with relatively little labor. The apparatus used in the test is shown in Fig. 2. When handled by a competent operator according to the prescribed schedule the method in its present form appears to yield reliable results.

The writer does not intend to convey the impression that these rather elaborate investigations were conducted with the idea in mind that mechanical analyses could be expected to furnish conclusive information as to the character of a given soil. It was fully recognized that other factors, such as grain shape, chemical and mineralogical composition, structure, density, and previous geological history, are of such importance that any analysis based on size alone will constitute at best only a rough indication of the properties of the material. Nevertheless, a grain distribution curve is of considerable assistance to an experienced soil analyst, and the writer feels that the time and effort expended in the investigations have been fully justified.

Limits of Consistency.—Among the many routine tests proposed for cohesive materials, the limit tests made by A. Atterberg[6] were considered of sufficient importance to warrant investigation. A sample of clay containing excess water acts like a liquid; it will not hold any definite shape unless restrained. If the water escapes by evaporation, the clay shrinks, loses certain of its fluid properties, and passes into the plastic state. It will now hold its shape, but offers little resistance to a change in shape. As further

[4] "Der Wiegnerische Schlämmapparat und seine praktische Anwendung," by H. Gessner, *Kolloid Zeitschrift*, Vol. 38, 1926.

[5] "The Hydrometer Method for Making a Very Detailed Mechanical Analysis of Soils," by G. J. Bouyoucos, *Soil Science*, September, 1928. References to previous publications will be found under this article.

[6] "Die Plastizität der Tone," von A. Atterberg, *Internationale Mitteilungen für Bodenkunde*, 1911, Heft 1.

evaporation takes place, additional shrinkage occurs, and the clay passes into a semi-solid state, in which an alteration of shape is accompanied by fracture. Finally, a point is reached at which the clay changes color and becomes relatively solid. Continued evaporation produces no shrinkage after this point is passed.

The change from one state to the next is gradual. In order to be able to define the various states, Atterberg set up and defined certain "limits" between them. The limit between the liquid and plastic states is called the "liquid limit"; that between the plastic and semi-solid, the "plastic limit"; and that between the semi-solid and solid, the "shrinkage limit." All the limit are expressed in terms of water content, that is, weight of water per unit weight of dry soil.

The shrinkage limit defines itself, and can be determined with comparative ease. Atterberg actually measured the change in length of a specimen during shrinkage. The method proposed by Charles Terzaghi, M. Am. Soc. C. E., is to compute the limit from a determination of the weight and volume of a sample wet (in the plastic state) and bone dry. If V_1 and W_1 represent the volume and weight, respectively, of the wet sample, and V_0 and W_0 those of the dry sample, the shrinkage limit is,

$$S = \frac{(W_1 - W_0) - (V_1 - V_0)}{W_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Apparatus has been developed for rapid and accurate determinations of the quantities involved.

The plastic limit is defined by Atterberg as the water content below which the material can no longer be rolled into threads about $\frac{1}{8}$ in. in diameter. Starting with the material in the plastic state, the test is performed by rolling a small sample into threads on an absorbent paper surface, thus causing the water content to decrease continually. When the sample refuses to form a thread, it is placed between watch glasses, weighed, dried, and weighed again. The loss in weight divided by the dry weight is the plastic limit.

The liquid limit was defined as the water content at which a groove through a pat of the soil about 1 cm. high, formed in an evaporating dish, would flow together at the bottom when the dish was tapped with the hand. It was noticed that determinations of this limit for the same soil by different operators varied considerably, due to the fact that each man had his own interpretation of the phrase "tapped with the hand."

In order to overcome these discrepancies, a series of experiments was conducted in co-operation with the U. S. Bureau of Public Roads. A simple apparatus was developed for performing the test mechanically, thus minimizing the personal equation. Fig. 3 is a view of the device ready for a test. The pat of soil is shaped and grooved with a special tool, shown lying on the table. Each turn of the crank lifts the dish and lets it fall through a height of 1 cm. Instead of trying to work the soil exactly to the liquid limit, it has been found advisable to determine, for several water contents, the number of blows required to close the bottom of the groove over a distance of $\frac{1}{2}$ in. The higher the water content, the fewer the number of blows will be. A

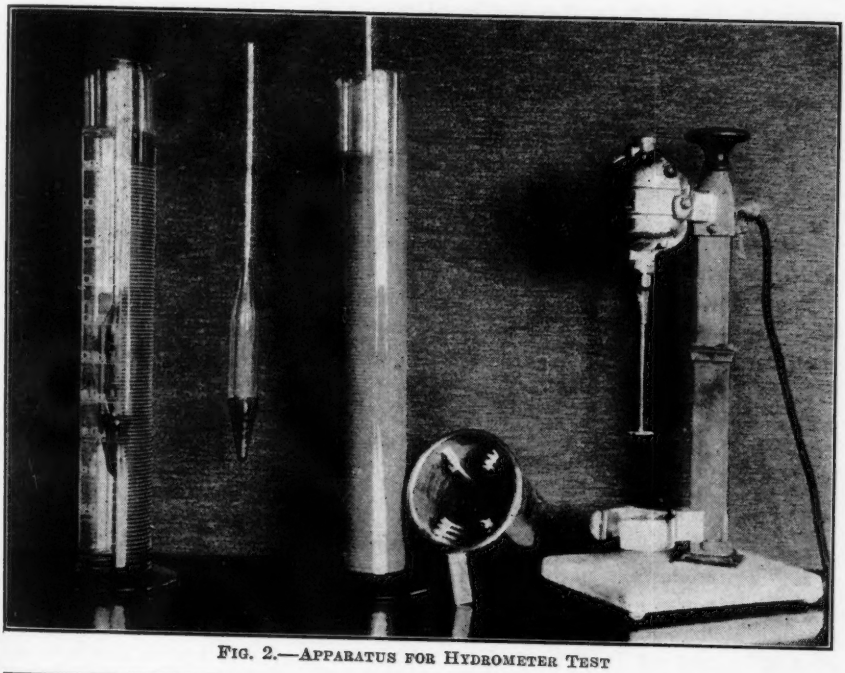

FIG. 2.—APPARATUS FOR HYDROMETER TEST

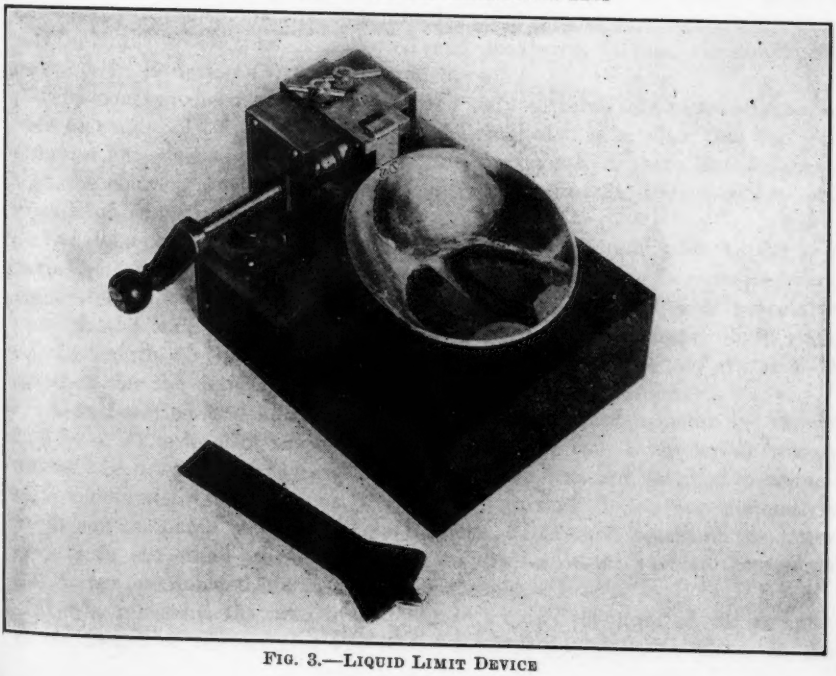

FIG. 3.—LIQUID LIMIT DEVICE

curve of water content against number of blows is plotted, and the water content corresponding to twenty-five blows, as read from the curve, is taken as the liquid limit.

Limit tests are in somewhat the same category as mechanical analyses, inasmuch as their results have only an indirect bearing on engineering problems. They are valuable, on account of their simplicity, in a rapid preliminary grouping of a set of soil samples prior to performing more elaborate tests.

Permeability.—One of the most important problems in soil mechanics is the study of the percolation of water through soils. In the normal case, with laminary flow throughout, percolation takes place in substantial accordance with Darcy's law, which states that the rate of percolation per unit area is directly proportional to the hydraulic gradient. The constant of proportionality is known as the "coefficient of permeability" of the soil. Symbolically, if Q represents the quantity flowing through an area of soil, A, in time, t, under a hydraulic gradient, i, Darcy's law may be represented by the equation,

$$Q = k\, i\, A\, t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, k is the coefficient of permeability.

One of the first pieces of apparatus developed in the laboratory was a constant-head permeameter. In this device the soil sample is contained in a glass cylinder, and provision is made for allowing water to percolate under a head which is kept at a known constant value throughout the test. The area and length of the sample are determined, and the water flowing through in a certain interval of time is collected and measured. Thus, all quantities except k in Equation (2) are determined.

The constant-head apparatus is suited only to relatively permeable soils, such as sands. In finer soils, the rate of percolation is so slow that the test occupies an extremely long period of time, during which there is the danger that the character of the soil may be altered by organic decomposition and growth of fungi.

For tests on fine-grained materials, it was found advisable to develop variable-head permeameters. In these devices the feed-water is contained in a stand-pipe of known area just above the sample. As the water percolates through the soil, the level in the stand-pipe drops. A single observation on the length of time required for the head to drop a certain distance is sufficient for the determination of the coefficient of permeability.

A half-section and half-elevation of one type of permeameter are shown in Fig. 4. The level of the tail-water in which the base is immersed is kept constant by a waste overflow. The apparatus is so designed that the resistance to flow offered by the stop-cock, the porous disk, and the various channels is negligible compared to that of the soil itself. Hence the height of the water surface in the stand-pipe, measured from the tail-water surface, represents the energy available to drive the water through the soil.

Let a represent the area of the stand-pipe; A, the area of the sample; L, the length of the sample; and h, the head at any time, t. Then, in the

next succeeding interval of time, dt, during which the head drops a distance, dh, conditions may be represented, according to Equation (2), by the relation,

$$-a\,dh = k\frac{h}{L}A\,dt \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

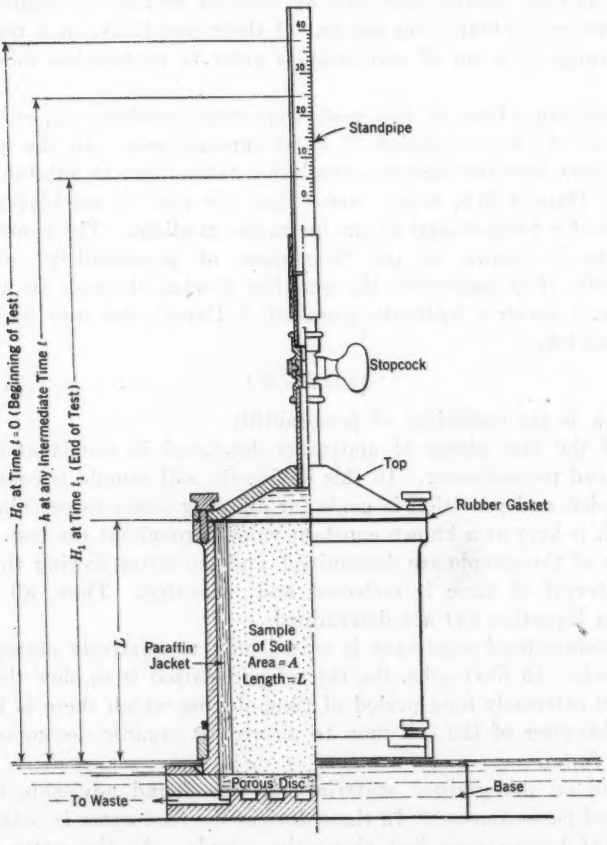

FIG. 4.—VARIABLE HEAD PERMEAMETER

If the measured head at any time, t_1, is H_1, and at some later time, t_2 is H_2, Equation (3) may be integrated between these limits, giving the following general value for the coefficient of permeability,

$$k = \frac{aL}{A}\,\frac{1}{t_2 - t_1}\,\log_e \frac{H_1}{H_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Thus, the coefficient may be evaluated from the observed data by a single computation.

By proper selection of the various component parts, apparatus of this type may be applied to a wide range of soils. The porous disks are commercially available in three grades, fine, medium, and coarse. If the soil

is too porous to permit the use of the coarse-grade disk, a perforated rubber disk covered with wire mesh may be used. The stand-pipe may be of any convenient size, ranging from a small pipette, for fine-grained soils, to graduated tubes $\frac{3}{4}$ in. in diameter for coarser materials. Very pervious sands may be tested by an extension of this method, in which the entire apparatus consists of a long glass tube with a screen across the bottom. The sample is placed in the lower part of the tube, and the upper part serves as a stand-pipe.

Another important application is found in the direct testing of the permeability of soils in their natural state. Methods have been developed whereby undisturbed samples may be removed from the earth and encased in paraffin at once, thus being well protected until the tests can be made. A portion of the sample may be placed in the permeameter in its original paraffin jacket, as indicated in Fig. 4, so that it is subjected to a minimum of handling in preparation for the test. The method is at present applicable only to soils which possess at least some degree of cohesion.

Measurements of permeability by observation of the rate of capillary saturation have received considerable attention. Extended studies were made on the relation between the permeability of a soil and the rate at which water is drawn up into a dry sample of the material contained in a vertical glass tube with its lower end immersed. As a result of these studies, the method was successfully applied to field control of the embankment material of an earth dam.[7]

Certain fundamental defects associated with the vertical capillarity test, which necessitate careful and extensive preliminary calibration, seem to disappear in the horizontal capillarity test, first proposed by Mr. Arthur Casagrande. The dry pulverized sample of soil is contained in a glass tube with

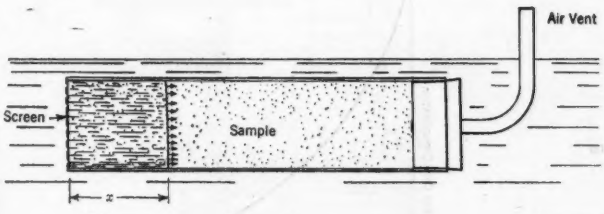

FIG. 5.—HORIZONTAL CAPILLARITY TEST

a screen at one end and a vented stopper at the other, as indicated in Fig. 5. The tube is immersed in a horizontal position in a shallow depth of water, and observations are made upon the rate at which the line of capillary saturation proceeds through the soil. If x represents the length of saturation in time, t, it may be shown theoretically that,

$$x^2 = t \sqrt{k F \frac{1+e}{e}} \quad \ldots\ldots\ldots\

A plot of observed values of x^2 against t is, therefore, a straight line, of which, $\sqrt{k F \frac{1+e}{e}}$ is the slope. The void-ratio, e (ratio of volume of voids to volume of solid) can readily be determined by measuring the volume and the dry weight of the sample. Thus, if the factor, F, is known, the coefficient of permeability, k, is readily found.

Experiments made on a wide range of soil types give a mean value of F of 100 000, when t is in minutes and k is in centimeters per second. Deviations from this mean are $\pm 50\%$ for ordinary soils. The value of F is smallest for very mixed-grained soils and largest for very uniform soils.

For a rapid approximate determination of the permeability, the result of a capillarity test, using a value, $F = 100\,000$, will in many cases be sufficiently accurate. For closer determinations, a series of calibrating tests should be made in order to furnish a value of F applicable to a particular range of soil types. The method appears at present to be reliable, and by far the simplest ever devised, especially for rapid field demonstrations.

Compressibility and Consolidation.— The compressibility of a soil is studied by measuring its change in volume produced by a change in load. Under the pressures usually encountered, the compression of the solid material is negligible, so that a decrease in volume of a soil mass under load is almost entirely due to decrease in the volume of voids. Results of compressibility tests are usually represented as curves of void-ratio (ordinates) against unit pressure (abscissas). The curves have a characteristic shape as shown in Fig. 6.

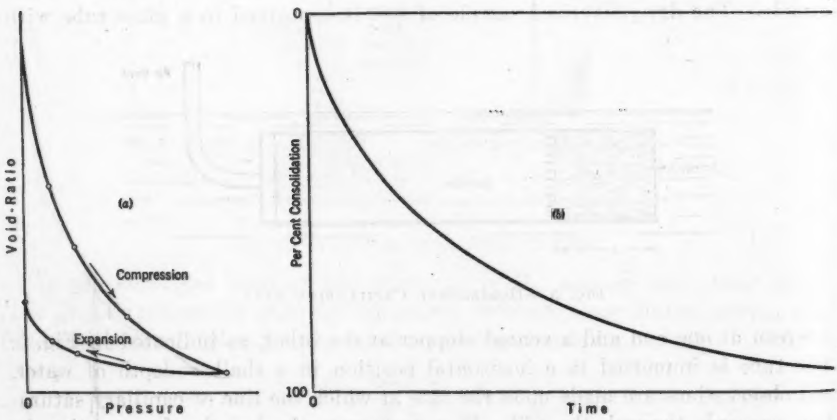

FIG 6.—TYPICAL DIAGRAMS OF CONSOLIDATION TEST

Dense, granular masses, such as compact sands, are the least compressible soils. Clays are characterized by a much higher compressibility. Dr. Terzaghi[8] has shown that the compression diagrams of sands and clays

[8] "Erdbaumechanik," pp. 86 and 90.

could be represented by the same logarithmic equations, the only difference being in the scale of void-ratio.

A rational explanation for the high compressibility of clays was furnished by a study of the compressibility of mixtures of sand and mica.[9] Both were granular, non-cohesive materials, and exhibited none of the characteristic properties of clay; yet it was found possible to select a mixture of sand and mica such that the compressibility characteristics of typical clays could be reproduced in detail. The researches showed definitely that the shape of the grains has an extraordinary influence on this important property of soils.

If a coarse, dry, granular material is placed under load in such a manner as to prevent lateral displacement, a decrease in void-ratio takes place almost instantaneously. A quantity of air corresponding to the change in volume is forced out, but there is practically no resistance to its escape. If the air is replaced by water, that is, if the mass is saturated, the change in volume is necessarily accompanied by a forcing out of some of the water. Since the resistance offered by coarse-grained soil to the passage of water is relatively small, compression is again almost instantaneous. If the soil is fine-grained, however, it may offer considerable resistance to the escape of the excess water. The compression in this case does not take place instantaneously, but occupies an appreciable period of time. The gradual compression of fine-grained, saturated soils under load is known as consolidation.

Consolidation tests are made with an apparatus which has been gradually developed from the primitive device first used by Dr. Terzaghi.[10] A half cross-section and half elevation of this instrument is shown in Fig. 7.

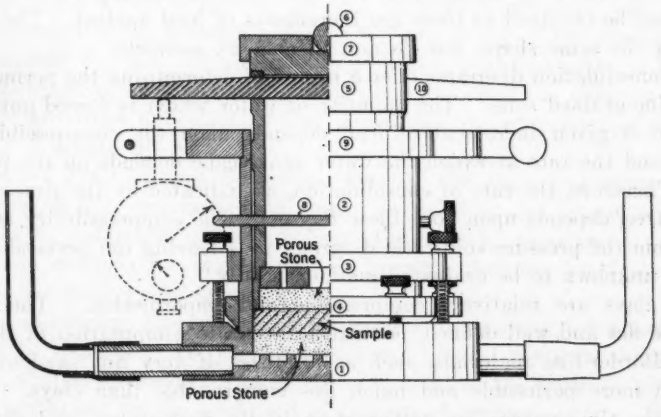

FIG. 7.—CONSOLIDATION APPARATUS

The soil sample is retained within a cylinder (2) and is compressed by means of load applied to a piston (5). The aligning pin (8) holds the piston immovable within the cylinder while the test is being prepared, and is

[9] "The Compressibility of Sand-Mica Mixtures," by Glennon Gilboy, Jun. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 555.

[10] "Erdbaumechanik," p. 83.

removed when the first load is applied. The amount of compression produced by a given increase of load is measured by two Ames dial gauges reading to 0.0001 in. Free passage of water at the two faces of the sample is provided by the porous disks in the base and cylinder. By filling the base stand-pipes and the interior of the piston with water, the test may be performed with the sample entirely submerged, so that a state of complete saturation exists at all times.

In performing the test, loads are applied in definite increments. While any given increment is acting, the dial readings are noted at convenient intervals of time. The dials are allowed to come to rest—that is, consolidation is allowed to proceed to completion—before the next increment is added. Depending upon the magnitude of the increment and the nature of the soil, the time required between increments varies from a few hours to one or two days.

After consolidation under the maximum load desired is complete, it is customary to measure the expansion of the soil by removing the load in increments. The final dial readings under each increment are translated into void-ratio, and the compression and expansion curves plotted as shown in Fig. 6(*a*).

The rate of compression under constant load, measured by the various intermediate dial readings taken at various times, may be conveniently represented by a curve such as that of Fig. 6 (*b*). The total compression under the given increment is taken as 100%, and the intermediate dial readings, expressed in percentage of total movement, are plotted against the corresponding values of time. Although only one curve is illustrated, actually as many curves may be obtained as there are increments of load applied. The curves are all of the same shape, but do not, in general, coincide.

The consolidation diagrams offer a means of determining the permeability of very fine-grained soils. The quantity of water which is forced out of the sample by a given increment of load depends upon the compressibility of the soil, and the rate at which the water can escape depends on the permeability. Therefore, the rate of consolidation, as indicated by the time-consolidation curve, depends upon both these factors. The compressibility is determined from the pressure-void-ratio diagram, thus leaving the permeability as the only unknown to be evaluated mathematically.[11]

True clays are relatively compressible and impermeable. Their time curves are flat and well defined, permitting accurate computation of the constants. Border-line materials, such as mixtures of very fine sand and silt, are much more permeable and much less compressible than clays. Consequently, the time curves are rather steep in the first stages, and the dials move so rapidly that accurate readings are difficult to obtain. To overcome this difficulty an arrangement has been devised whereby direct permeability tests can be performed on the sample while the compression test is going on. One of the stand-pipes in the base of the apparatus is closed, and the other is connected to a vertical pipette tube hung above the device in such a

[11] "Principles of Final Soil Classification," by Charles Terzaghi, M. Am. Soc. C. E., *Public Roads*, Vol. 8, No. 3, May, 1927.

manner that water from the pipette flows into the base, up through the sample, into the piston, and out of the holes previously occupied by the aligning pin. The action is analogous to that of the variable-head permeameter previously described.

The combined consolidation and permeability test furnishes a means of checking the accuracy of the permeability coefficients computed from the consolidation curves. The close agreement obtained in two tests selected at random is shown in Fig. 8.

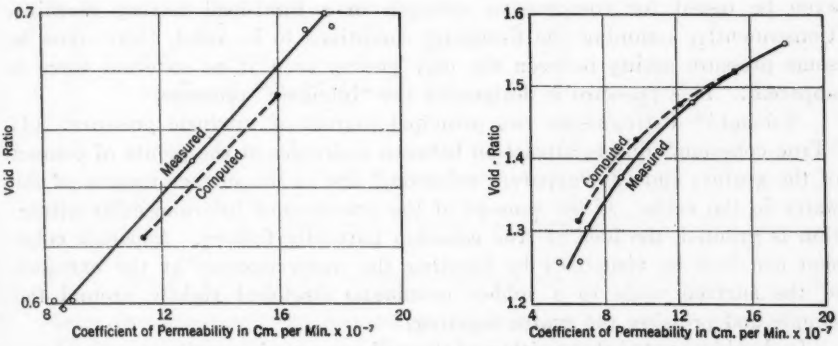

FIG. 8.—COMPARISONS OF DIRECT AND INDIRECT PERMEABILITY DETERMINATIONS

Inasmuch as the consolidation test yields information upon the amount of compression which a soil sample will undergo under load and upon the rate at which this compression proceeds, it ranks as one of the most important soil tests ever devised. In addition to its purely scientific value, it has many practical applications, among which may be mentioned the analysis of settlements of buildings underlaid by clay strata, and the consolidation characteristics of hydraulic-fill cores.

Internal Friction and Cohesion.—One of the most important physical properties of soil, especially with regard to stability and lateral pressure, is the shearing resistance. Attempts have been made by numerous investigators to develop methods for evaluating this property, but the problems involved are still far from a final solution.

The underlying conceptions of the nature of shearing resistance are not difficult to visualize. If two soil grains are pressed together, the resistance against sliding of one grain on another is equal to the pressure between the grains multiplied by the coefficient of friction of grain on grain. The shearing strength of an accumulation of grains is the sum of a great number of such individual sliding resistances. Therefore, in order that a mass of soil shall exhibit shearing strength, two conditions must be fulfilled: (1) There must be pressure between the grains; and (2) there must be a definite coefficient of friction between the grains. If one of these quantities is zero, the mass behaves like a liquid, no matter what the value of the other factor may be.

A familiar illustration of this statement is furnished by the so-called "quicksand" phenomenon. The dynamic energy of a current of water flowing upward through a mass of sand tends to counterbalance the downward force of gravity. If the energy of the water is sufficiently great to counteract entirely the action of gravity, the effective weight of each individual sand particle is zero. Hence, the intergranular pressures are reduced to zero, and the mass behaves essentially like a liquid.

On the other hand, a stiff clay exhibits the properties of a solid. A cylindrical specimen cut out of the mass retains its shape unaided, and may even be tested for compressive strength in a light-load testing machine. Consequently, assuming the foregoing conditions to be valid, there must be some pressure acting between the clay grains, even if no external force is apparent. This pressure is designated the "intrinsic" pressure.

Terzaghi[12] distinguishes two principal sources of intrinsic pressure: (1) "True cohesion," due to attraction between molecules at the points of contact of the grains; and (2) "apparent cohesion," due to the surface tension of the water in the voids. If the concept of the existence of intermolecular attraction is granted, the idea of true cohesion naturally follows. Apparent cohesion can best be visualized by likening the water menisci at the entrance of the surface voids to a rubber membrane stretched tightly around the sample and pressing the grains together.

It should be noted that neither of these phenomena is peculiar to clay alone. They are usually associated with clay, because their effects are most pronounced in very fine-grained material; but they can occur to a certain degree in any type of soil. For example, a moistened sand can be moulded into a cylinder, because the surface tension of the water produces some apparent cohesion. If the cylinder is immersed in water, the surface tension disappears, and the cylinder disintegrates. True cohesion also exists in a mass of sand, but its magnitude is vanishingly small on account of the relatively small number of points of contact per unit area.

True cohesion depends upon a great number of factors, such as grain size, grain shape, structure, density, and previous geological history of the deposit. It also appears to be influenced by chemical factors and electrolytic effects about which relatively little is known. Apparent cohesion is a function of many such variables as those mentioned, and, in addition, is subject to wide variations produced by changing moisture conditions. Thus, it is fairly evident that considerable time and study are required before the complications involved in the shearing resistance of soils can be properly understood.

The first method used at Massachusetts Institute of Technology in the study of these effects was based upon tests of the compressive strength of clay cylinders. When the sample fails in compression, shear cracks become visible on the surface, as indicated in Fig. 9(*a*). According to Mohr's theory,[13] the angle, α, between two such shearing planes is $90° - \phi$, wherein, ϕ is the angle of internal friction of the material. With the compressive

[12] "Erdbaumechanik," p. 94.

[13] "Technische Mechanik," by Otto Mohr, Ernst, Berlin, 1906.

strength, p_c, also determined from the test, it is possible to construct a rupture diagram for the specimen, as shown in Fig. 9(*b*). The intrinsic pressure is represented by the distance marked p_i. The vertical intercept marked c is the cohesion as defined by Coulomb. Thus, it is theoretically possible from this test to determine the important strength constants of the material.

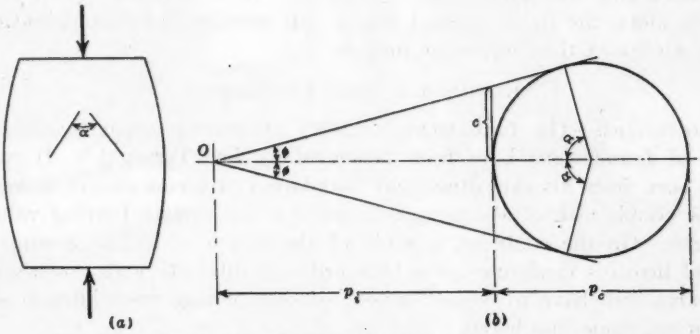

FIG. 9—RUPTURE OF A CYLINDER ACCORDING TO MOHR'S THEORY

Many investigators have endeavored to measure the shearing resistance of soils by subjecting samples to direct shearing stress. An important contribution in this line was made under Dr. Terzaghi's direction in 1929. Under a co-operative agreement with the New England Power Construction Company, a large and elaborate shearing device was constructed, based on the principle of applying a known vertical pressure to the soil sample, and measuring the amount of horizontal force required to produce failure in shear. A brief description of the apparatus and an outline of the results of the first series of tests are available in published form.[14]

Subsequently, a second machine was constructed, embodying certain of the principles of the first device, but designed to be simpler, more accurate, and more flexible. A close-up of one of the shearing units and a general view of the apparatus are shown in Figs. 10 and 11. A distinct advantage of this machine is that six units may occupy the table simultaneously, whereas only one rig for producing horizontal shear is required.

The apparatus has been in constant use since its construction, and has yielded new and valuable information upon various types of soil. It is impossible to formulate any general conclusions from the data at present available. Certain interesting indications, however, may be enumerated tentatively. A sand in a dense state exhibits a definitely higher angle of friction than the same sand loose. The difference appears to decrease with decreasing grain-size. Shear failure in sand is accompanied by marked deformations of the mass normal to the failure plane. Saturation does not decrease the angle of friction of sand. The clays thus far tested exhibit considerably higher angles of friction than were expected. In stiff clays possessing high true cohesion it is difficult to develop the full shearing strength of the material;

[14] *Engineering*, June 13, 1930.

the soil appears to fail progressively, due to local concentrations of high stress, which in turn are due to the inability of the material to undergo sufficient plastic yield to equalize the stress distribution. If this hypothesis proves to be correct, it may have an important bearing upon the question of the stability of slopes cut into soil of this character.

Opinions as to the physical nature and effect of shearing resistance in soils have been, and still are, widely divergent. It is believed that continued research along the lines outlined herein will provide useful contributions to the knowledge of this important subject.

Research in Soil Engineering

Foundations.—The fundamental aspects of bearing capacity and settlements of foundations have been presented by Dr. Terzaghi.[15] It may be readily seen from his exposition that foundation problems cannot be resolved into the simple task of assigning to a given soil a certain bearing value per unit area. On the contrary, a study of the action of soil as a supporting material involves mechanics of a high order of difficulty and a vast amount of research will have to be performed before anything resembling a simple solution can come into being.

Studies of the load-settlement characteristics of footings resting on sand have shown that even in this material wide variations are to be expected. The same sand exhibits quite different properties according to whether it is loose or dense. Under loads applied to the surface of a loose sand, a settlement of ½ in. was produced by a load of about ½ ton per sq. ft.; when the same material was tested in a compact state, the same settlement occurred under a load of about 5 tons per sq. ft. Results of this type indicate the need of a more accurate classification of sand, especially with regard to density, than is usually found in foundation codes. Corroborating previous evidence, the tests showed that the settlement under a given unit load, not too near the ultimate, is practically independent of the size of the loaded area.

A promising method for field exploration of sand deposits was successfully tried on the Houston-Essex Subway, in New York City, in co-operation with the contractors, the Corson Construction Corporation. The variations in the force required to press a steel cone into the soil were measured, and the results were correlated with a series of loading tests performed at various levels. The penetration device appeared to be very sensitive to changes in the density of the deposit, and these changes were also reflected in the loading tests. Similar investigations made later at the Massachusetts Institute of Technology, under controlled conditions, agreed very well with the field explorations. A distinct advantage of the method is that it permits of testing the soil in place, thereby avoiding the errors which are associated with sampling operations. Descriptions of the apparatus and procedure and outlines of the results thus far obtained have been published.[16]

[15] "The Science of Foundations—Its Present and Future," by Charles Terzaghi, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 270.

[16] "Die Tragfähigkeit von Pfahlgründungen," by Charles Terzaghi, M. Am. Soc. C. E., *Die Bautechnik*, 1930, Heft 31 u. 34; and "Penetration Tests Give Bearing Power of Deep Subsurface Soils," by George Paaswell, M. Am. Soc. C. E., *Engineering News-Record*, April 2, 1931.

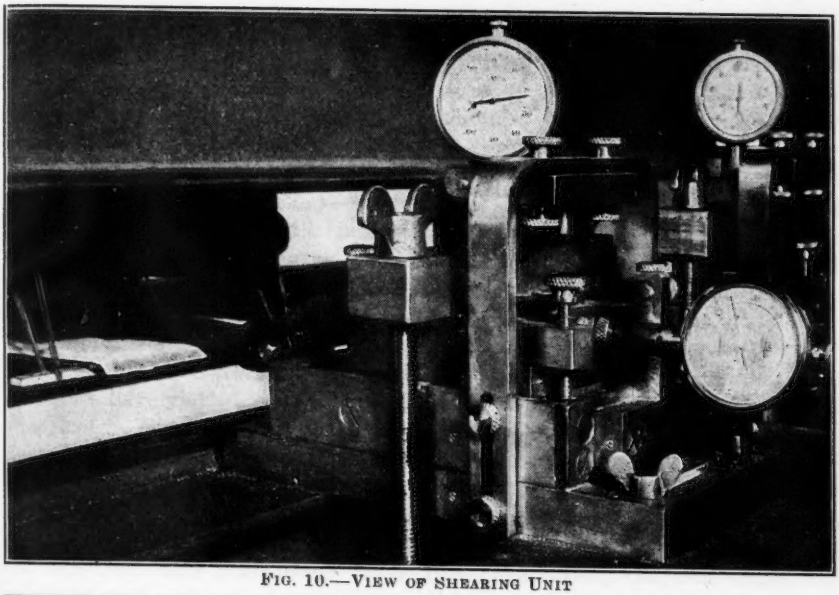

FIG. 10.—VIEW OF SHEARING UNIT

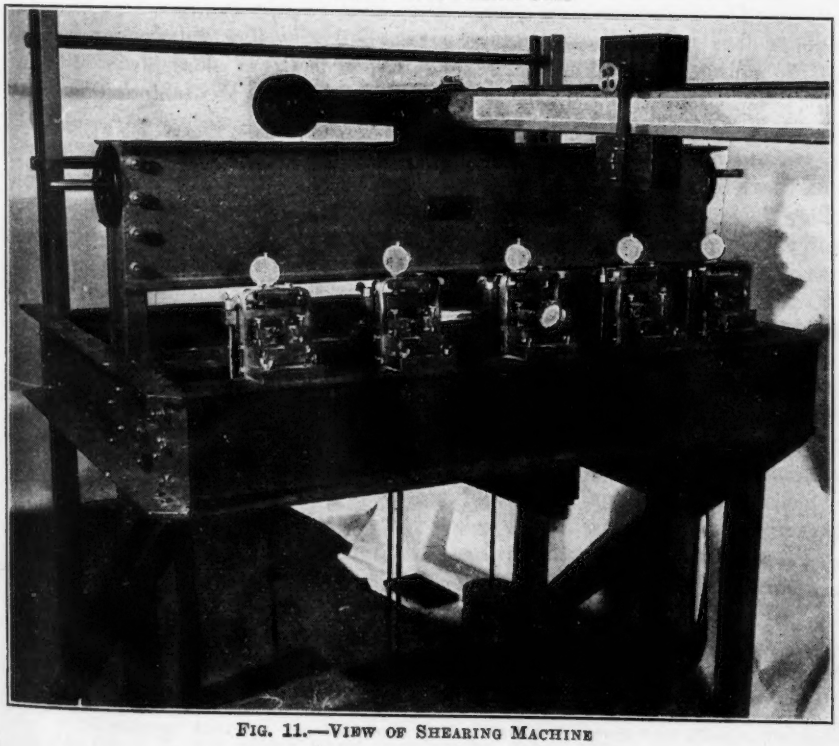

FIG. 11.—VIEW OF SHEARING MACHINE

A practical application of the theory of consolidation is found in the analysis of the settlement of buildings underlaid by deep strata of clay. The basic idea of the analysis is simple. If the stresses produced in the soil by superimposed loads can be computed, and the deformation that the soil will undergo under a given stress is known, it should be possible to determine the settlement of the structure, both in magnitude and in rate. Actually, the problem is very complex. As to the stress distribution, while the classical theory of Boussinesq is the best available, it can yield only approximate results. The analysis of the stress-strain characteristics of soils thus far performed includes only the simplest cases. Most important of all, some assumption as to homogeneity of the underground must be made, and this is only approximately fulfilled, even under the most favorable circumstances.

With all these drawbacks, however, it is encouraging to know that such analyses have yielded valuable information. An interesting case was studied in 1929.[17] A group of buildings founded on piles had settled to an alarming extent, notwithstanding the fact that the load applied to the piles was very low, compared with the rated bearing capacity. Borings disclosed the presence of a layer of mud about 120 ft. below the surface. A stress analysis was made, neglecting the presence of the piles, and consolidation tests on samples of the mud were performed. With this information, the settlements and rates of settlement of various parts of the group were computed. The results agreed remarkably well with the observed values, showing that such studies are of considerable practical importance in obtaining a conception of the causes, amounts, and rates of foundation settlements.

More recent investigations have established the vital importance of performing tests on samples in an undisturbed state. It had been previously assumed that if a sample were thoroughly remoulded to a liquid state and subjected to pressure, the previous geological history of the material would be reproduced on a laboratory scale. It seems, however, that this is not by any means the case. Apparently, the natural structure, once destroyed, can never be duplicated, and even in ordinary varieties of clay there seems to be sufficient structural effect so that the consolidation characteristics in the natural and the remoulded states are quite different.

In order to provide means for obtaining large undisturbed samples of cohesive material, a new type of rotary drill, operating without water, was designed. It has been used successfully in sinking a 6-in. hole through fill, sand, and clay to a depth of nearly 100 feet. The undisturbed samples obtained from this operation are being utilized in an elaborate settlement analysis. The tools are in process of improvement, with a view to bringing the cost down to a point somewhere near to that of wash borings. This work is being carried out under the auspices of the Committee on Earths and Foundations of the Society.

Progress along these lines would be greatly facilitated if every construction job were looked upon as a large-scale experiment. The data required are comparatively easy to collect. A schedule of loads, a series of bench-

[17] "Settlement Analysis—the Backbone of Foundation Research," by Charles Terzaghi, M. Am. Soc. C. E., *Proceedings*, World Eng. Congress, Tokyo, 1929.

mark observations taken at various times, a geological profile of the underground, and a few soil tests on the different strata will furnish the material required for a settlement analysis. The additional cost of such observations would be a negligible item in the total cost of construction; the information obtained would be of highest value.

Hydraulic-Fill Dams.—In co-operation with the Miami Conservancy District, a thorough investigation of the properties and behavior of hydraulic-fill dams was started in the summer of 1927. A vertical shaft, 5 ft. in diameter, lined with concrete, was sunk to a depth of 80 ft. into the core of the Germantown Dam. Using tools especially designed for the purpose, undisturbed samples of the core material, 4 in. in diameter and 12 in. long, were removed by hand from time to time as excavation proceeded. The samples were sealed into iron containers, carefully packed, and shipped to the laboratory. Undisturbed samples, in 2-in. pipes, 2 ft. long, sealed at both ends, were removed from a drill hole near-by. Many unforeseen difficulties impeded the progress of the field work, and the District is to be commended for its persistence in mastering these difficulties and for the thoroughgoing manner in which it fulfilled its part of the program.

The experimental work included extensive tests on the physical properties of the soil, with emphasis on permeability, consolidation, and compressive strength tests. The tests showed that the average degree of consolidation of the core was about 25% at the time the samples were removed. This does not mean that the material was soft or yielding. On the contrary, it was very resistant—due to its relatively high angle of internal friction (about 27°)—and exhibited a compressive strength varying from 1 kg. to 2 kg. per sq. cm.

A mathematical analysis of the rate of consolidation of the core was also undertaken. The theory is extremely complicated, and the equations finally obtained present a forbidding appearance. It was found possible, however, to obtain from these equations a comparatively simple method of solution. Using this method, an investigator can construct a curve showing the progress of consolidation with time. The curve for the Germantown Dam is reproduced in Fig. 12. It is interesting to note that the dam was about seven years old when the samples were removed, and that the theoretical percentage consolidation corresponding to seven years is 25%, which checks exactly the results of the experimental investigations. It will be still more interesting to take samples from the core at the end of another ten or twenty years, to see whether the theory is still corroborated.

As studies on the general problem of the stability of hydraulic-fill dams progressed, it became increasingly apparent that a major rôle is played by the cohesionless, permeable outer sections of the dam, which the writer suggests might be designated by the term "shell." The impression was obtained that the emphasis usually placed upon the composition and behavior of the core had tended to obscure the functions of the shell to a degree incommensurate with their importance. As far as the core is concerned, the worst

that can happen is that it does not consolidate at all. If the shell is sufficiently strong to hold the core in place under these circumstances, the structure should be stable.

In accordance with this idea, an attempt was made to evaluate the stress conditions in the shell with the core assumed to be in a liquid state. Mathematical analysis leads to rather complicated equations, but it was found possible to develop a graphical solution whereby the position of the weakest plane through the shell and the angle of friction which the shell material must develop in order to insure stability, can readily be determined. From the standpoint of safety, therefore, it appears that studies of the

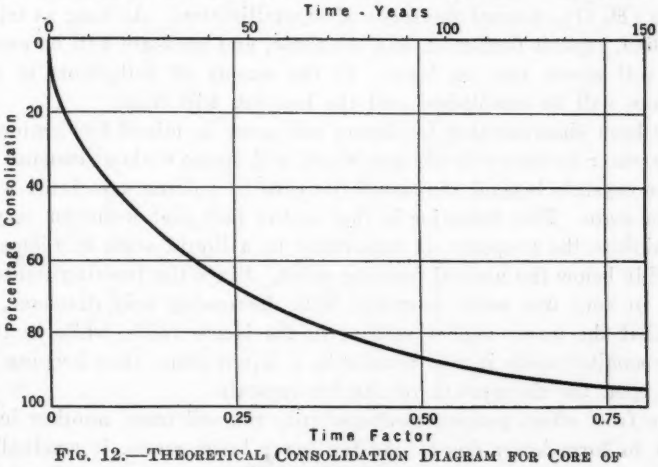

FIG. 12.—THEORETICAL CONSOLIDATION DIAGRAM FOR CORE OF GERMANTOWN DAM

physical properties of the shell material, with a view to selecting proper constituents and designing an adequate section, are even more important than the tests usually made on the material of the core.

Frost Heaving of Highways.—Studies of frost action on highways were conducted in co-operation with the U. S. Bureau of Public Roads and the State Highway Department of New Hampshire. The purpose of the experiments performed at the Massachusetts Institute of Technology was to study frost action under controlled conditions.

The experimental unit consisted of a wooden box, 3 ft. square and 3 ft. deep. On one side of the box was an observation well, 1 ft. square, communicating with the interior of the box at the bottom. A gravel filter was placed in the bottom of the box and well, thick enough to cover the opening between them. In this manner, the water level within the box could be controlled from the well, because the gravel prevented any soil from entering. The sides of the box were coated on the inside with a thin layer of grease, over which were laid small overlapping sheets of Manila wax paper. The object of the grease and paper was to minimize the retarding effect of friction between the soil and the sides of the box. The soil to be investigated was carefully tamped into the box, filling it level full. On top was placed a slab, 2 ft. 8 in. square, of concrete or macadam, representing the road surface.

Eight such units were buried to full depth in the ground, and observations on water level, depth of frost, heaving of the slabs, etc., were continued throughout the winter of 1928-29. Difficulties were experienced in holding the water levels constant; the boxes appeared to leak continuously, in spite of all precautions. Nevertheless, the experiment was successful in general, and paved the way for more detailed investigation of the phenomenon.

The researches showed that frost heaving cannot occur to any appreciable extent unless there is a continuous supply of water from below the frost level. The pressures associated with freezing water are due to the growth of ice crystals in layers which, in turn, is due to the travel of the large trihydrol molecules $(H_2O)_3$, toward the focus of crystallization. As long as trihydrols are supplied, crystal formation will continue, and pressure will be exerted to lift the soil above the ice layer. If the supply of trihydrols is cut off, equilibrium will be established and the heaving will cease.

It has been observed that ice layers will grow in mixed fine-grained soils, while the water in the voids of clean sands will freeze without causing growth of the ice crystals beyond the usual increase in volume associated with the change in state. This behavior is due to the fact that water in very small spaces exhibits the property of remaining in a liquid state at temperatures considerably below the normal freezing point. Since the freezing temperature of water in very fine voids decreases with decreasing void diameter, it can be seen that the water will crystallize in the larger voids, while in the surrounding smaller voids it will remain in a liquid state, thus keeping supply channels open for the growth of the ice crystals.

As the frost effect penetrates deeper into the soil mass, another ice layer will start to form lower down. As the lower layer grows, it gradually cuts off the water supply, so that further growth of the upper layer is prevented.

The frost effect is thus shown to be controlled by three factors: First, the position of the ground-water level, or of veins and pockets of water which may serve as a source of supply for excess ice accumulation; second, the void-size distribution of the soil in which freezing occurs; and, third, the capillary properties of the soil between the source of water supply and the frost level.

Earth Pressure on Retaining Walls.—The most elaborate research project undertaken at the Soil Mechanics Laboratory was a large-scale experimental study of the lateral pressure exerted by earth behind a retaining wall, conducted in co-operation with the New England Power Construction Company. The project was laid out in detail by Dr. Terzaghi in December, 1928, and construction was started the following month. The apparatus was ready for operation in the latter part of April.

An isometric sketch of the device is shown in Fig. 13. The soil to be tested is contained in a concrete bin, 14 ft. square and 10 ft. high. The retaining wall, forming one side of the bin, consists of a structural steel frame, *F*, faced with a concrete slab, *K*. By means of counterweights, *Q*, at each end of the wall, the center of gravity of the entire wall assembly is made to lie in the face of the slab. The wall is suspended from hangers,

V_1 and V_2, the points of suspension being also in the face of the slab. Thus, when it is free, the wall hangs in a strictly vertical position; and any vertical component exerted by the earth on the face produces a direct pull on the hangers. The hangers are supported by scales above, which, in turn, rest on Columns C_1 and C_2 and on structural members supported by heavy columns

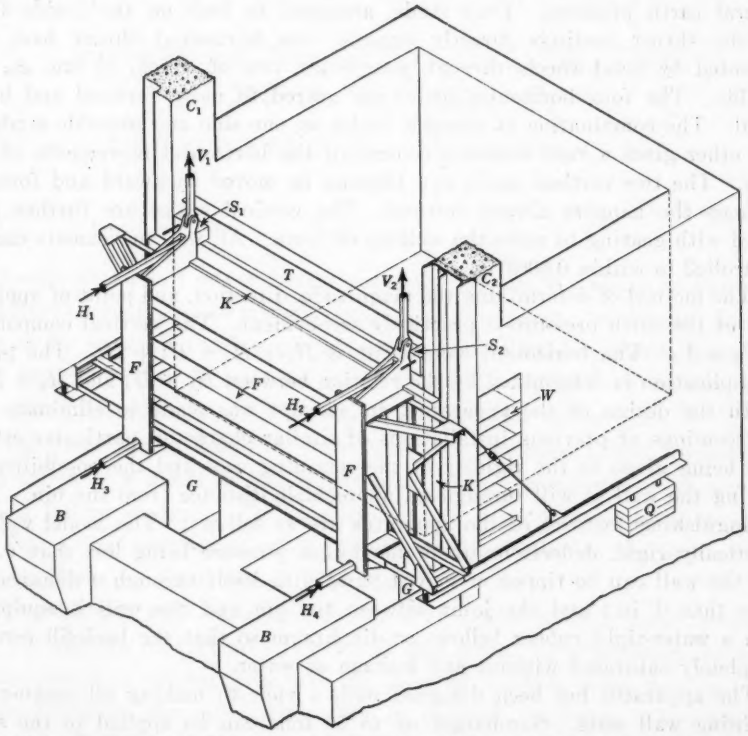

FIG. 13.—APPARATUS FOR STUDYING LATERAL EARTH PRESSURE BEHIND RETAINING WALLS

running up from the projecting foundation walls. The weight of the wall assembly is compensated by counterweights within the scales, so that the sum of the scale-beam readings is the vertical component of the earth pressure. The wall assembly weighs approximately 14 tons; yet the scales are so sensitive that they will record the weight of a man standing on the wall, with an error of $\pm$ 2 lb.

The horizontal pressures on the wall are balanced by thrust rods at the four corners, H_1, H_2, H_3, and H_4. The magnitudes of the thrusts are recorded by four scales. The two lower scales rest on concrete bases, B, in the substructure; the upper scales are supported by a rigid structural frame tied to the bin, the frame being supported by the same columns which carry the rear ends of the vertical scales. Thus, the bin, wall, and scales are included in a rigid unit which is independent of the building enclosing the apparatus. The unit is supported by a heavily reinforced concrete raft, 3 ft. thick, rest-

ing on fifty piles, 20 ft. long. The space above the raft and below the floor-slab of the bin forms an inspection gallery, *G*.

A window, *W*, of 1-in. plate-glass in a structural steel frame, is inserted in one wall of the bin to allow inspection of the earth within. The tie-beam, *T*, is introduced to prevent excessive deflections of the bin walls under the lateral earth pressure. Four studs, arranged to bear on the inside faces of the thrust castings directly opposite the horizontal thrust bars, are operated by hand-wheels through gear-boxes, two of which, S_1 and S_2, are visible. The four horizontal scales are geared to move forward and backward. The combination of movable scales on one side and movable studs on the other gives a very accurate control of the horizontal movements of the wall. The two vertical scales can likewise be moved backward and forward to keep the hangers always vertical. The vertical scales are further provided with gearing to move the wall up or down. All wall movements can be controlled to within 0.0001 in.

The method of determining the magnitude, direction, and point of application of the earth pressure is practicaly self-evident. The vertical component is $V_1 + V_2$. The horizontal component is $H_1 + H_2 + H_3 + H_4$. The point of application is determined by the relation between $H_1 + H_2$ and $H_3 + H_4$.

In the design of the apparatus, an attempt was made to eliminate the shortcomings of previous installations of similar character, particular attention being given to the rigidity of the retaining wall and the possibility of moving the wall at will through an appreciable distance from the bin. The distinguishing features of the apparatus are as follows: The model wall is practically rigid, deflections under maximum pressure being less than 0.001 in.; the wall can be tipped or moved parallel to itself through a distance of more than 1 in.; and the joint between the bin and the wall is equipped with a water-tight rubber bellows or diaphragm so that the back-fill can be completely saturated without any leakage of water.

The apparatus has been designed with a view to making all manner of retaining wall tests. Surcharges up to 50 tons can be applied to the surface of the back-fill. The effects of different slopes of the surface of the back-fill can be measured. The action of filters and drains, as well as the effect of partial and complete saturation, can be studied. A brief summary of the results of the first series of tests has appeared in published form.[18]

Acknowledgments

In addition to the acknowledgments and references embodied in the text of this paper, the writer wishes especially to mention the work of Mr. Arthur Casagrande, of the U. S. Bureau of Public Roads. Together with other important contributions, Mr. Casagrande has been chiefly responsible for the development of the liquid limit device, the hydrometer analysis, the horizontal capillarity test, the latest type of shearing device, and the frost-heaving experiments. Much valuable assistance has been obtained from the work of students, whose researches have been presented as theses to the Faculty of the Massachusetts Institute of Technology.

[18] *Engineering*, May 30 and June 13, 1930.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ENGINEERING FEATURES OF THE ILLINOIS WATERWAY [1]

By Walter M. Smith,[2] M. Am. Soc. C. E.

Synopsis

Locks, dams, and power houses along the Illinois Waterway in the State of Illinois constitute the primary subjects of this paper. A general description of the structures at Lockport, Brandon Road, Dresden Island, Marseilles, and Starved Rock, is followed by a theoretical analysis of culvert valve chambers, operating machinery, etc., as designed for this project.

The steps in the design of miter gates are reviewed briefly, including a description of a metal sealing-spring plate which permitted a more accurate and a lighter design. The gate anchorages were planned so as to make possible a fine adjustment of the rotation axis, similar to the arrangement for the locks at Sault Ste. Marie, Mich.

Interference with existing structures at Lockport, Ill., made necessary the installation of vertical lift-gates at this point. Furthermore, an emergency gate was necessary to take the place of movable gates that regulated water levels at the other dam sites in case of accidents. The physical features of all these units are treated herein.

An important part of the paper is an analysis of stresses in the side walls of the locks where the entry of the culverts complicates the design. The problem is solved by the Principle of Least Work, and one complete numerical example is given.

Introduction

The canal officially known as the Illinois Waterway, extends from the down-stream end of the Sanitary District Canal at Lockport, Ill., to Starved Rock, a distance of about sixty miles. The plan and profile of the canal are shown in Fig. 1.

[1] Presented at the meeting of the Waterways Division, Milwaukee, Wis., July 11, 1929. Written discussion on this paper in **January, 1932**, *Proceedings*.

[2] Asst. Chf. Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

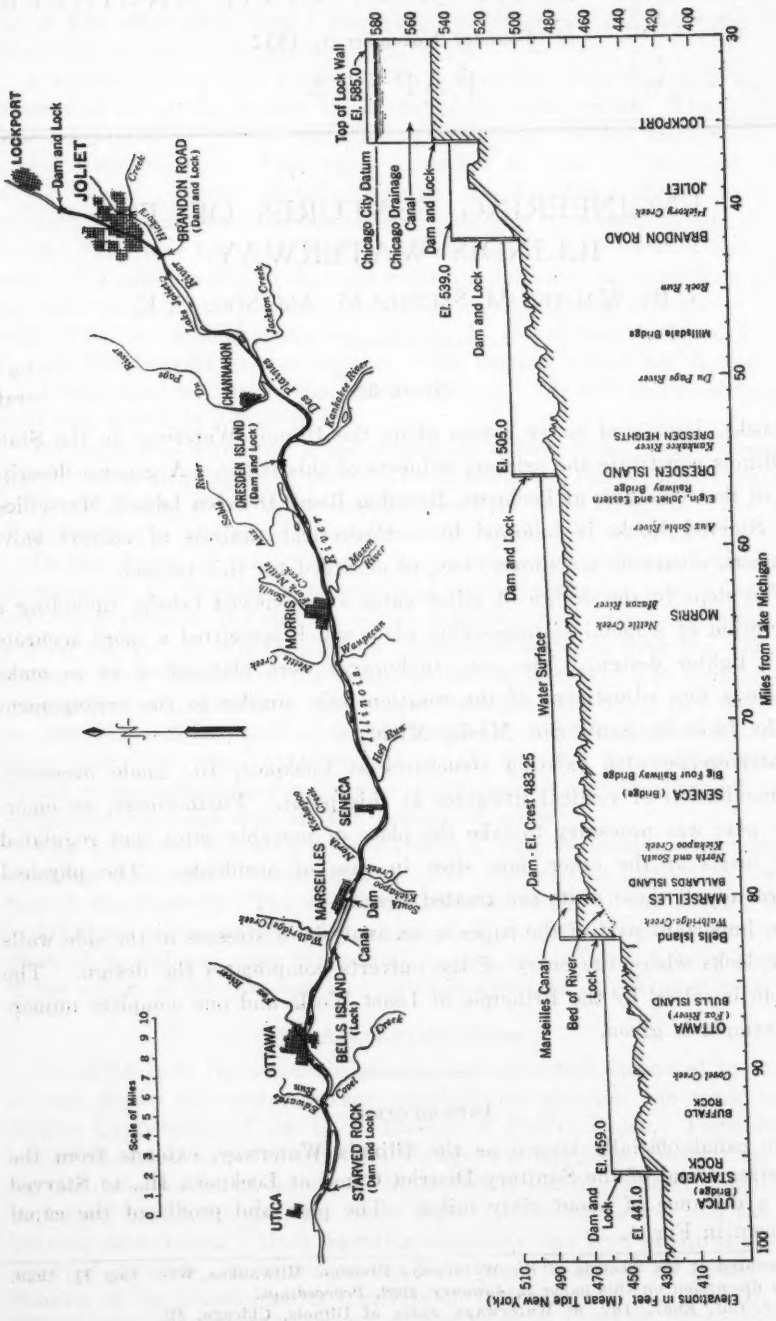

FIG. 1.—GENERAL PLAN AND PROFILE, ILLINOIS WATERWAY

Locks have been located at Lockport, Brandon Road, Dresden Island, Bell's Island (about 2½ miles below Marseilles and known as the Marseilles Lock), and at Starved Rock. Dams and head-gate structures for power houses have been constructed at Brandon Road, Dresden Island, Marseilles, and Starved Rock. The Sanitary District of Chicago had a dam and power house in operation at Lockport before the beginning of work on the Illinois Waterway.

The head-gate structures of the power houses, except at Marseilles, were built first as a part of the dam, so that the power-house construction could be left for a later date. For a distance of 2½ miles up stream from Marseilles Lock, the Waterway consists of a canal entirely separate from the river. This is the case also for a distance of 2 miles down stream from Lockport Lock. Otherwise, the Waterway consists of a series of pools, dredged to a proper depth and width wherever necessary. The dimensions established were a minimum width of 200 ft. and a minimum depth of 9 ft.

Description of Structures

Lockport Lock.—The structure a short distance southwest of Lockport consists of a lock with a maximum lift of 40 ft., which has miter-gates at the lower, and vertical lift-gates at the upper, end. An emergency dam has been constructed at the up-stream entrance to the lock.

Brandon Road Pool.—Des Plaines River enters the canal between Lockport and Brandon Road. Hence, there will be much greater flood flow at this site than at Lockport, and provision has been made to take care of a flood of 35 000 sec-ft. It is desired to hold this water at a very uniform level (Elevation 539.0) because this is the pool elevation through Joliet, Ill. A concrete retaining wall was built along the east side of the river through Joliet from the upper end of the city, southward 13 500 ft. to the dam, because the water level is much higher than the lower parts of the surrounding land. On the west side of the river a concrete wall was built from Jackson Street to the dam, a distance of 11 500 ft. As the retaining wall along the east side had to be built in any case, about 1 200 ft. of the lower end of it was constructed as a spillway weir. The crest of this weir is 2 ft. 3 in. below the water level as maintained by a series of twenty-one Taintor gates, 2 ft. 3 in. high by 50 ft. long. A gate has been constructed for passing ice, similar to the one at the Lockport Power Plant, of the Chicago Sanitary District.[3]

The crest of the long spillway weir is at Elevation 536.75, New York mean tide datum; the crest of the Taintor gates is at Elevation 539; and the crest of the ice-chute weir, at Elevation 534. A study was made to see if it would be better and more economical to construct several "Stickney" siphon spillways[4] rather than the long ogee weir. It was found that in order to secure a considerable flow with very slight rise, the siphon spillways were slightly more economical; but there was a serious disadvantage in the fact that there is almost no increase in flow in these spillways as the water rises

[3] *Engineering News*, Vol. 60, 1908, p. 512.
[4] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1098.

FIG. 1.—GENERAL PLAN AND PROFILE, ILLINOIS WATERWAY

in the reservoir, whereas in the ogee type the capacity increases as the three-halves power of the head. For a 3-ft. head on the ogee weir, for example, the flow would be about 5.2 times as great as for a head of 1 ft. and, for a 4-ft. head, it would be 8 times as great. Because this great increase in discharge for a small increase in head was very desirable, it offset decidedly the slight economy of the siphon spillway. In addition, six sluice-gates with clear openings of 6 by 8 ft. were provided in the base of the dam. The normal lower-pool elevation is about 506.2, making the normal lift of the lock, 32.8 ft. The section of canal from Lockport to Brandon Road (see Fig. 1) is about 6 miles long.

Intercepting sewers are constructed in the base of the retaining walls at Joliet, the one on the east intercepting all the sewage and storm-water run-off from that part of the city and discharging it below the dam. The west side of Joliet is much higher than the east side, except for a narrow strip along the back of the retaining wall. Therefore, most of the storm-water run-off is conveyed into the canal through submerged cast-iron pipes that pass through the wall. The sewage from the west side is conveyed into the intercepting sewer in the base of the wall. A 36-in. pipe has been placed across the river bed to convey this sewage to the larger intercepting sewer in the wall on the east side near its down-stream end.

Dresden Island Pool.—From Brandon Road to Dresden Island Dam is about 14 miles and two rivers of consequence enter this stretch, the Du Page from the north and the Kankakee, with a very large drainage area, from the south. According to the best data available the maximum flood flow that will occur at this site is 75 000 sec-ft. The dam consists of a concrete weir with nine Taintor gates, 60 ft. long—each sustaining a head of 14 ft. of water normally—and an ice chute, 30 ft. long, with a gate sustaining a head of only 5 ft. when closed. This gate is similar to the ice chute for the dam at Brandon Road.

The elevation of the normal upper pool and the Taintor gates is 505; the crest of the Taintor-gate weirs is at Elevation 491; the crest of the ice-chute weir is at Elevation 500; and the normal lower-pool elevation is 486.3.

Marseilles Pool.—From Dresden Island to Marseilles Dam is about 24.5 miles; but no rivers of consequence enter the canal between these two points so that the maximum flood flow for this site is 77 000 sec-ft. The Marseilles Lock is about $2\frac{1}{2}$ miles below the dam and a canal has been excavated beside the river from the dam to the lock instead of using the river. A short distance down stream from the old dam, the new one, 970 ft. long, is planned to extend to the side of the canal entrance. It includes seven Taintor gates, each 60 ft. long and 14 ft. high. The tops of the Taintor gates were set at Elevation 483.25.

Marseilles Canal and Lock.—A canal separate from the river extends about $2\frac{1}{2}$ miles from the Marseilles Dam to the Marseilles Lock. Since this canal is intended both for navigation and power purposes the power house is near the lock. The general plan and sections of the lock are shown in Fig. 2, while Fig. 3 shows the details at the upper gates.

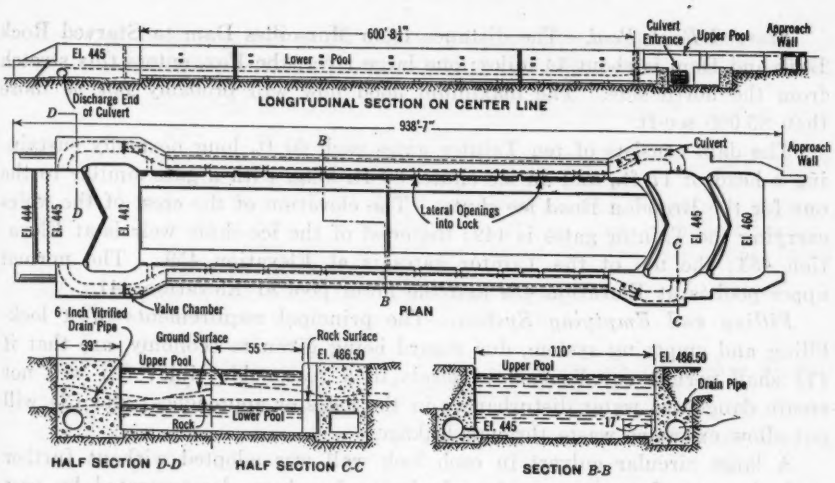

FIG. 2.—GENERAL PLAN AND SECTIONS, MARSEILLES CANAL AND LOCK

All the locks on the Illinois Waterway have a usable length of 600 ft. and a width of 110 ft., with lifts varying from 15 to 40 ft. Miter-gates will be used at both up-stream and down-stream ends of locks, except at Lockport, where a different type of gate had to be used at the up-stream end for reasons to be given subsequently.

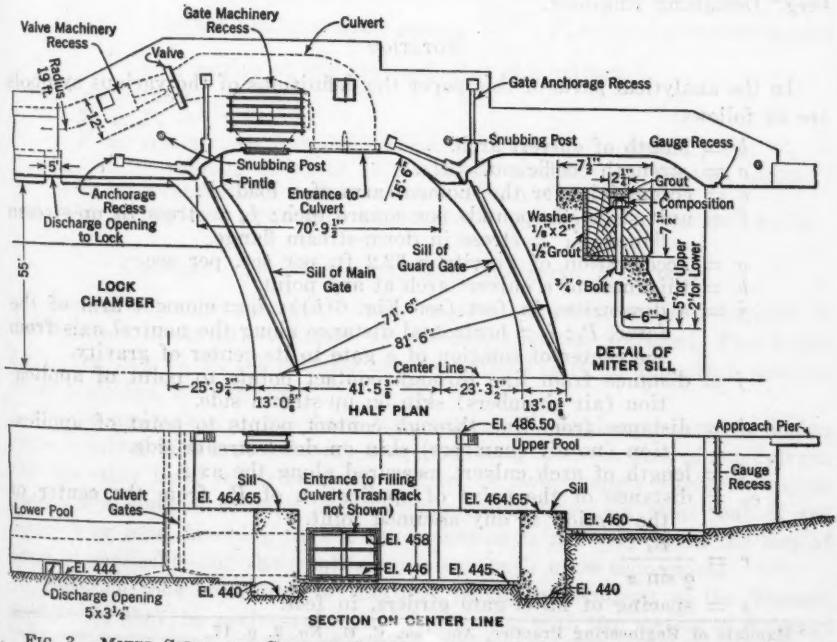

FIG. 3.—MITER SILLS AND CULVERT CONNECTIONS AT UPPER END OF MARSEILLES LOCK

Starved Rock Pool.—The distance from Marseilles Dam to Starved Rock Lock and Dam is about 14 miles; one large river, the Fox, enters this stretch from the north side. The maximum flood flow will probably not be more than 85 000 sec-ft.

The dam consists of ten Taintor gates each 60 ft. long normally sustaining a head of 17 ft., and an ice chute, 30 ft. long, with a gate similar to the one for the Brandon Road ice chute. The elevation of the crest of the weirs carrying the Taintor gates is 442; the crest of the ice-chute weir is at Elevation 454; the top of the Taintor gates is at Elevation 459. The normal upper pool is at Elevation 459 and the lower pool at Elevation 441.

Filling and Emptying System.—The principal requirements of a lock-filling and emptying system, due regard being given to economy, are that it (1) shall permit equalization of levels in a reasonable time; (2) will not create dangerous water disturbances in the locks or approaches; and (3) will not allow excessive waste through leakage.

A large circular culvert in each lock wall was adopted without further study because the economy of such design has been demonstrated by past experience. Studies showed that two culverts, 12 ft. in diameter, would ensure equalization of levels in from 7 to 12 min., depending on the location of the lock and subject to the limitation in equalization of 4 ft. vertically per min. A complete description of the filling and emptying system and lock valves has been written by L. D. Cornish, M. Am. Soc. C. E.,[5] and the operating machinery for the valves has been described by Mr. A. S. Grossberg,[6] Designing Engineer.

Notation

In the analytical parts of this paper the definitions of the various symbols are as follows:

b = length of culvert arch.
c = strength coefficient = Hs.
e = eccentricity, or the moment arm of a load, P.
f = unit stress, in pounds per square inch; f_u = stress in up-stream flange; f_d = stress in down-stream flange.
g = acceleration of gravity = 32.2 ft. per sec. per sec.
h = thickness of a culvert arch at any point.
i = a dimension, in feet (see Fig. 6(*b*)); i_1 = moment arm of the force, P_2; i_2 = horizontal distance along the neutral axis from the center of rotation of a gate to its center of gravity.
j = distance from line through contact points to point of application (air chambers) skin on up-stream side.
j' = distance from line through contact points to point of application (no air chambers) skin on down-stream side.
l = length of arch culvert measured along the axis.
e_n = distance of the point of application of P_n from the center of the section at any assumed joint.
$r = \dfrac{P_1}{2 \sin \alpha}$.
s = spacing of miter-gate girders, in feet.

[5] Manuals of Engineering Practice, Am. Soc. C. E., No. 3, p. 17.
[6] *Loc. cit.*, p. 21.

t = time required to fill a lock chamber, in seconds.
w = weight, in pounds; w_g = weight of miter-gate; w_w = weight of 1 cu. ft. of water.
x = transverse horizontal distance from the center of rotation of a gate to its center of gravity.
y = horizontal distance from the down-stream face of a gate to its center of gravity.
A = area, in square feet; A_c = combined area of the pair of culverts leading into a lock chamber; A_l = area of lock chamber; A_n = area of a culvert section at any assumed joint, acted upon by the force, P_n.
A_1 and A_2 = pull in the anchorages with gates closed.
C = a constant, varying from 0.75 to 1.15.
E = modulus of elasticity.
F = total stress, in pounds; F_u = stress in the up-stream flange; F_d = stress in the down-stream flange.
H = head, or lift, in feet.
I = moment of inertia of a section of any assumed joint that is acted upon by P_n.
K = resisting force, is equal and opposite to $\frac{P_1}{2}$.
L = distance from a culvert support, measured along the axis; dL = an infinitely small distance along the axis.
M = bending moment in a culvert section at any assumed joint.
N = resisting force, is equal and opposite to P_1.
P = direct pressure, in pounds; P_1 = total water pressure perpendicular to flange on one girder; P_2 = total water pressure against ends of one girder; P_3 = total pull exerted on anchors, A_1 and A_2, at the top of a miter-gate = $\frac{W_g i^2}{i_1}$; P_n = total pressure normal to a section at any assumed joint of a culvert.
r = radius, in feet.
T = $P_1 + P_2$.
V = velocity, in feet per second, in up-stream approach; and V_t = velocity of water at the throat section of a Venturi meter.
W = total work of resistance in a culvert wall.
α = angle of miter-gate with a line perpendicular to the lock-walls.

Culvert Valve Chambers and Operating Machinery

The culvert valve chambers were designed according to the principles of the Venturi meter as developed[1] by the late Clemens Herschel, Past-President, Am. Soc. C. E., the culverts being circular (12 ft. in diameter), and the valve openings, 9 ft. square.

For some reason it has been the practice of canal engineers to design culvert valves with a considerably larger opening than the area of the culvert. On the other hand it has been the practice of water supply engineers to put gates in a conduit or pipe line of considerably less area than that of the conduit or pipe, knowing that if the reduction is not very great the loss of head is negligible and the smaller gates are much more economical.

Herschel took advantage of this fact in his development of the Venturi meter. In 1887, he performed many experiments with two meters of different

[1] *Transactions*, Am. Soc. C. E., Vol. XVII (1887), p. 228, and Vol. XVIII (1888), p. 133.

sizes. The first was with a tube 1 ft. in diameter and a throat diameter of $\frac{1}{3}$ ft., so that the throat area was one-ninth the area of the tube. In the second set the tube was 9 ft. in diameter, with a throat diameter of 3 ft., making the throat area one-ninth the area of the tube, the same as in the other case.

The loss of head of the water passing from the inlet to the throat may be expressed in two ways: First, in terms of velocity at the throat, V_t; and, second, in terms of increase in velocity at the throat, $V_t - V$.

In the experiments with the small meter, with velocities at the throat varying from 10 to 35 ft. per sec., the loss of head in terms of the velocity at the throat was $0.00172\ V^2_t$ and in terms of the increased velocity at the throat, $0.00218\ (V_t - V)^2$.

In the case of the large meter the loss of head was slightly higher, probably due to the fact that it was not so well made; also, there was a greater variation in values, the loss of head being about $0.00230 V^2_t$, or $0.00290\ (V_t - V)^2$. In the case of the small meter a very uniform coefficient of 3.0 was obtained, but in the larger size the coefficient was about 2.5 and varied considerably more than in the small one. In these experiments the velocity at the throat of the meter was nine times as great as that in the tube.

In designing the inlet gates in the main gate-house at the Ashokan Dam, of the Catskill Water Supply System for New York City, advantage was taken of the Venturi principle by the Engineers of the Board of Water Supply. The valve, which is the throat of the meter, is 5 ft. in diameter, with an area of 19.6 sq. ft.

The sides flare on the down-stream side at an angle of 2° 30′, the same as in the Herschel meters, to a diameter of 7 ft. $4\frac{1}{4}$ in. and an area of 42.6 ft. at a distance of about 29 ft. from the valve. This gives an increase of 117% in area over the throat area. In testing these valves a coefficient of 1.18 was obtained before the Venturi action was broken.

In the filling and emptying system of the locks of the Illinois Waterway the culverts are circular, 12 ft. in diameter, with an area of 113 sq. ft. The valves are 9 ft. square. A transition section down stream becomes a circle 12 ft. in diameter in a distance of 19 ft. from the valve, or an increase in area of 40 per cent. For the same distance, the increase in area of the tail-tube in the Ashokan valve is 72 per cent. It is hoped that as high a coefficient will be obtained in the valves as that obtained in the Ashokan tests.

The formula for finding the time required for filling a lock chamber is,

$$t = \frac{2\,A_l\,\sqrt{H}}{C\,A_c\,\sqrt{2g}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and it has been the practice to take $C = 75$. It was found in testing the Panama Canal locks after completion that when the valve area was much greater than the culvert area, $C = 0.83$. Using this value in Equation (1) the time required to fill the Marseilles Lock, if the rise is unlimited and $H = 21.6$ ft., is: $t = \frac{2 \times 74\,910\,\sqrt{21.6}}{0.83 \times 226\,\sqrt{64.4}} = 464$ sec., or 7 min. 44 sec.

Assuming that a coefficient of $C = 1.15$ will be obtained in the valves constructed in this manner and substituting this coefficient and the area of the valves, $A_c = 162$, for the area of the culverts in Equation (1), the time required will be 467 sec., or an increase of 3 sec. in the time of filling the lock due to using the smaller size of valves. This shows that with this size and type of valve the time of filling and the time required for a vessel to pass through the locks will not be materially increased by using the smaller valves unless the coefficient falls below 1.15 which is not at all likely.

If a coefficient of 1.10 only should be obtained in the valves, the increase required in filling one lock is only 25 sec.; the increase in time of the boat going through the five locks is less than 3 min.

The gate-valves are raised 10 ft. in about 1 min. The machinery is driven by a 10-h.p. electric motor, at 900 rev. per min., through a worm-wheel drive, and a train of gears revolving a drum, to lock the gate in any position of opening or closing. A close-link chain, attached to the gate, is deflected from a vertical direction by an idler sheave near the top of the coping. It runs a short distance horizontally through a hawse pipe and winds around the scored drum. The machinery can be operated by hand from the top of the masonry by a windlass with a sliding bevel-gear pinion that can be engaged very quickly by means of a detachable collar on the shaft. For a more complete description of the culvert valves see Lock Valve Manual No. 3, previously mentioned.[8]

Miter Lock Gates

With the exception of the upper gates for the structure at Lockport all lock gates are of the mitering type. There will be thirteen miter-gates of two leaves each, the total weight of which will be about 3 000 tons. The leaves are 65 ft. long, 7 ft. wide, and vary in height from 20 to 59 ft., and in weight from 90 to 320 tons.

Structurally each leaf consists of several horizontal girders rigidly connected to a miter post on the free end and a heel post on the hinge end. The leaf is stiffened vertically by five sets of diaphragms between all girders, and has diagonal tension rods with turnbuckles on the up-stream side to provide means of adjusting the miter-posts to a vertical position. The miter and quoin-posts are built of structural steel shapes as shown in Fig. 4, the web-plate being in sections which extend well in between the girder flanges as vertical diaphragms which transfer the thrust of the leaves along the quoin and miter-posts to the web-plates in the horizontals. The sheathing or skin-plates are on the down-stream side. Buckle-plates were utilized where possible and they will be in tension except where a double skin was used to form air chambers. Intercostals are not necessary with buckle-plates; hence, this type of sheathing costs less than flat plates.

No other type of gate was considered because mitering gates have long been the standard form for canals and have given excellent satisfaction in operation and maintenance. Estimates based on actual designs have shown that usually, for the lock as a whole, the most economical rise of mitered

[8] Manuals of Engineering Practice, Am. Soc. C. E. No. 3, p. 18.

gates is approximately one-fourth the lock width and that, for a leaf 65 ft. long, the economical width of the horizontal girder is about 7 ft. The flanges should be straight except for a short curved length near each end of the up-stream flange.

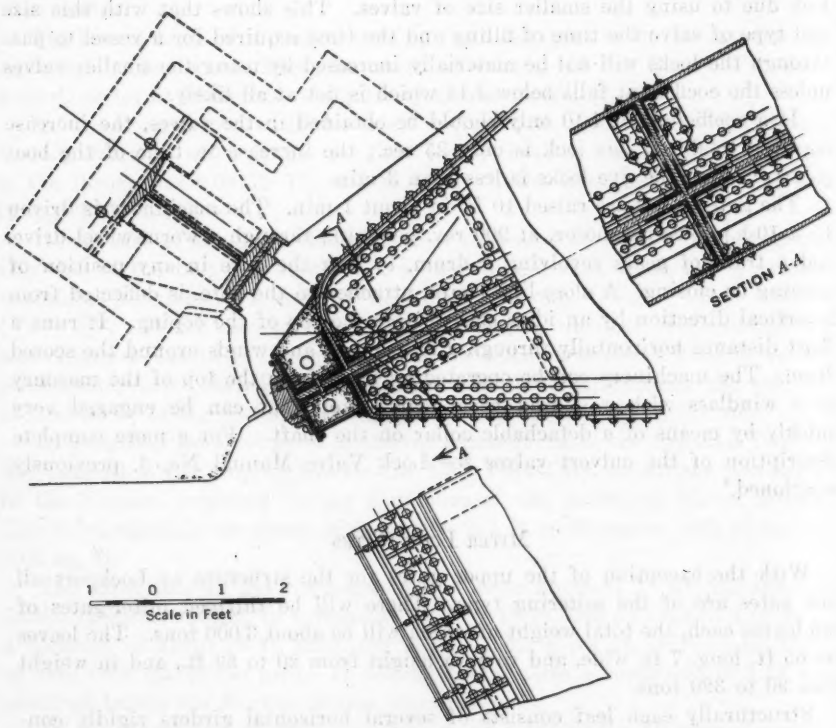

FIG. 4.—QUOIN POST DETAILS, ILLINOIS WATERWAY

Air chambers were provided in all miter gates in order to utilize their buoyant effect to reduce the working reactions on the pintles and hinge-pins. These air chambers are formed by the addition of sheathing plates on the up-stream side. For this purpose buckle-plates were used, except along the curved portion, where flat plates supported by intercostals were necessary.

The bearing plates along the quoin and miter-posts are of steel with convex curved surfaces. The bearing plates in the hollow quoin or reaction post are also of steel, but with a concave surface, the radius of which is slightly greater than the convexity radius of the quoin-post. The bearing plates are accurately adjusted with set screws after which melted lead-antimony alloy is poured into the spaces left for that purpose behind the plates.

The determination of the stresses in the gate and gate anchorage is not described at length in this paper because it would involve merely a repetition of what has been written before on this subject.

It was originally thought that air chambers would not be needed in any of the miter-gates except those previously mentioned, but after both the upper and lower gates at Marseilles Lock had been completed according to the original designs, and after making operating tests, it was found desirable to construct air chambers in all the gates to relieve the heavy load on the pintles. Fig. 5 shows the various details.

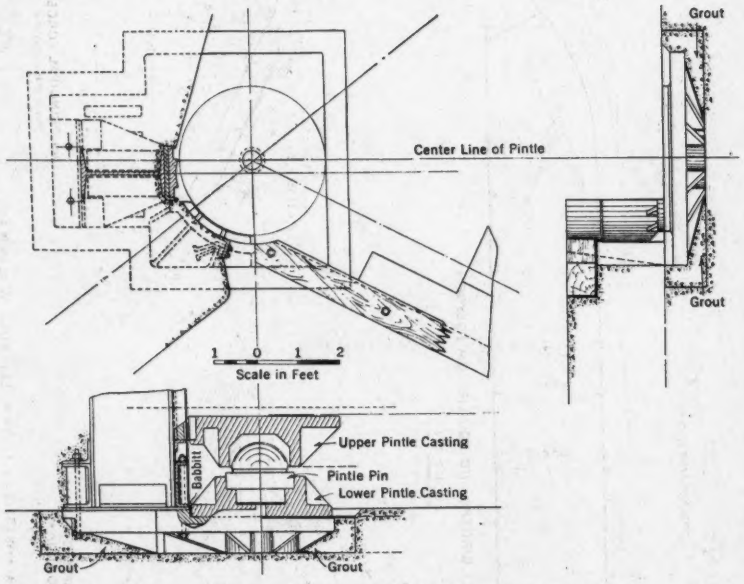

FIG. 5.—DETAILS OF PINTLE, ILLINOIS WATERWAY

A graphic analysis of the principal forces acting on a gate and the equilibrium polygon and flange area curve for a girder are shown in Fig. 6. In the cases illustrated (Fig. 6(*a*)), the strength coefficient, $c = Hs = 100$; $f =$ 14 000 lb. per sq. in.; and $w_w = 62.5$ lb. A sample computation to determine the flange area, in square inches, at the center of a structure is as follows:

$$\text{Flange area} = \frac{Tj}{7f} = \frac{71.8 w_w H s \times j}{7f} = \frac{71.8 \times 100 \times 62.5 \times 7.78}{7 \times 14000} = 35.6, \text{ in}$$

which, 71.8 was measured from Fig. 6(*c*); and $w_w H = c = 100$. To determine the value of $j = 7.78$ ft., refer to Fig. 6(*d*). Taking moments about O:

$$j - x = \frac{0.112\,N + 3.55\,P_2 + 32.456\,K - 16.228 \times 0.5\,P_1}{P_1\ P_2} = 7.78$$

and, similarly,

$$j' = \frac{0.112\,N + 32.456\,K - 16.228 \times 0.5\,P_1}{P_1} = 8.226$$

The total stresses, in pounds, in the flanges then are equal to:

$$F_u = \frac{Tj}{7} \text{ and } F_d = \frac{T(j-7)}{7}$$

FIG. 6.—DESIGN DATA FOR MITER GATES, ILLINOIS WATERWAY

Fig. 6(*b*) shows that the maximum pull on one anchor is, for gate closed, $-0.84\,P_s$ and, for gate open, $+0.12\,P_s$, and for the other anchor, $1.03\,P_s$ maximum with gate nearly open.

On the diagrammatic cross-sections of the various gates for the waterway are shown the girder spacing and the coefficient, *c*, of each girder. The water pressure diagrams in Fig. 7 show the actual or assumed head acting on each

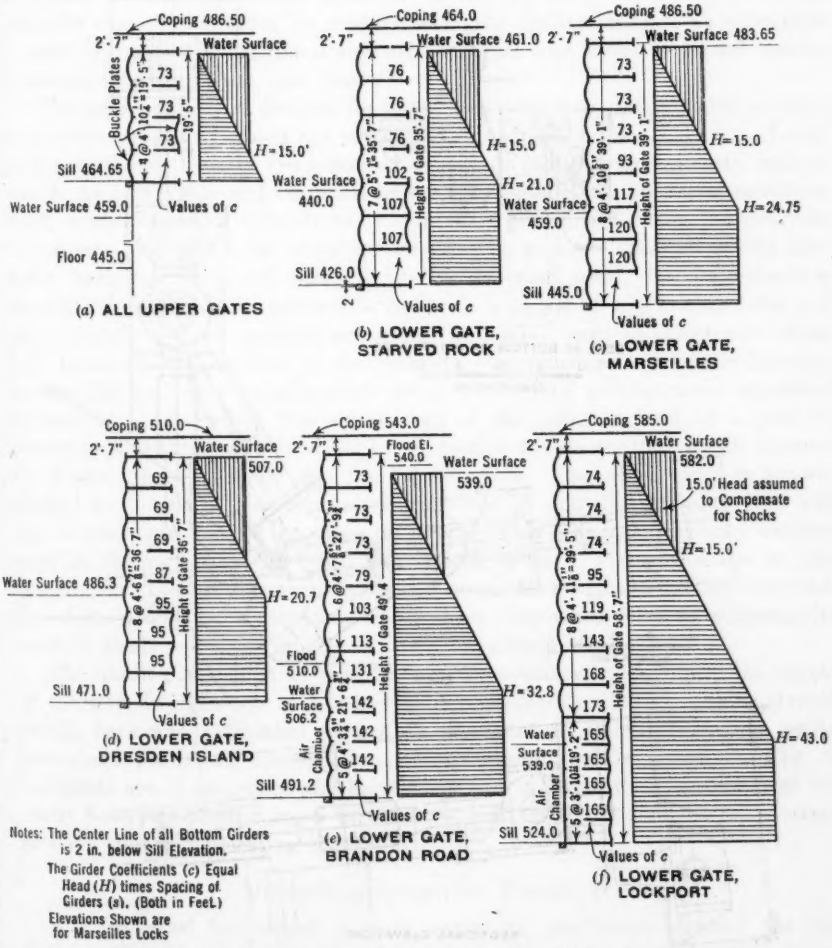

FIG. 7.—WATER PRESSURE DIAGRAMS FOR MITER GATES, ILLINOIS WATERWAY

leaf and the coefficients, $c = Hs$, for each girder. The load per linear foot on any girder equals $w_w\,c$. As the required cross-sections of girders are directly proportional to their coefficient (Fig. 6(*c*)), a correct determination of flange areas for a girder the coefficient of which is, say, 100, solves the flange-area requirements of all girders. It should be noted that the total flange stresses

are less when the skin is on the down-stream side than when it is on the up-stream side. The difference is due to the elimination of the force, P_2, shown for the left-hand leaf in the analysis of "Principal Forces (Fig. (6*d*))."

Figs. 6 and 7 were devised by L. D. Cornish, M. Am. Soc. C. E., Chief Engineer of the Division of Waterways, State of Illinois, and were of great value in the designing of all miter-gates.

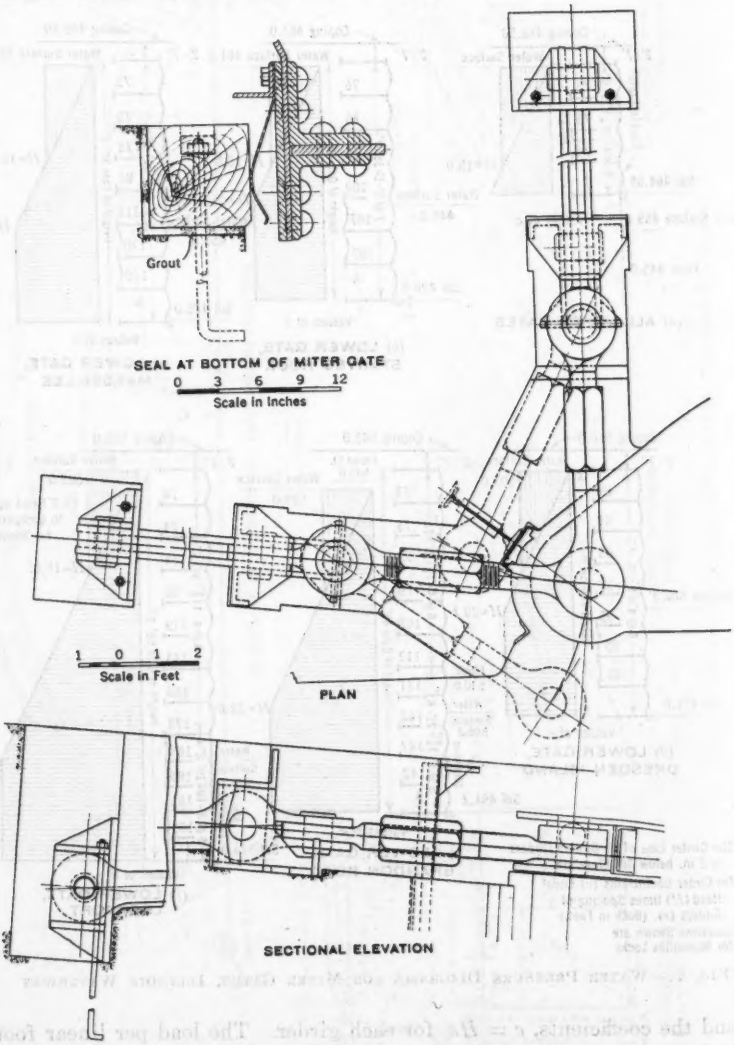

FIG. 8.—ANCHORAGE OF MITER-GATE, ILLINOIS WATERWAY

Contact at Miter Sill.—In the design of mitering gates it has been the practice heretofore to permit close contact and heavy bearing between the

lower gate girders and the miter-sill. The result of such design does not permit an accurate and scientific design of either the lock-gate or the miter-wall. Both must be designed unnecessarily strong to withstand the unknown and indeterminate pressures, which are assumed to be heavy. This indeterminate condition has been remedied by placing a metal sealing-spring plate on the gate, which alone will be in contact with the miter-sill when the gate is closed, and which cannot transmit to the sill more than a part of the pressure due to the water in contact with the spring itself. This permits a more accurate and lighter design of both gate and miter wall and results in decidedly lower costs (see Fig. 8).

The gate anchorage designs for these locks differ in many respects from any other designs of which the writer has knowledge. (See Fig. 8.) Large gate anchorages usually are extended deep into the massive concrete against which the gate abuts and are made of eye-bars or built-up structural members. Such anchorages are difficult to set and hold accurately while the concrete is poured. To secure the same results, that is, to carry the pull of the gate when hanging free, well down into the concrete mass, the gate-reaction monoliths of these locks are reinforced by a network of reinforcing bars and only straight eye-bar anchors are used, the anchor ends of which are about 4 ft. below the coping level of the concrete. Troughs are left in the concrete so that the bars may be accurately set at leisure and subsequently embedded in concrete. A rather fine adjustment of the rotation axis of a gate is necessary and this is provided for by adjustable double eye-bars which connect the hinge pin of the gate and the fixed anchor. The adjustment is accomplished by means of sleeve nuts with threads of slightly different pitch and size at each end, but in the same direction. This design is the same as that used in the new locks at Sault Ste. Marie, Mich. The advantage of this design over the ordinary turnbuckle is a much finer adjustment per turn and, therefore, a less force required, which is an important item as adjustments must be made when the anchor is under maximum stress.

The quoin post which is embedded in the concrete to distribute the thrust of the gate is made up of I-beams with pressure-distributing angles riveted on the back and a channel section on the front to which the hollow quoin plates are fixed. A cross-section of the quoin post is shown in Fig. 5. The posts are in one piece and the bottom end is bolted to a projection of the pintle bed-plate casting so as to facilitate holding the quoin post in an exact position during concreting operations.

Miter-Gate Operating Machinery

The so-called bull-wheel type of operating machinery devised for the Panama Canal is used to open and close the miter-gates (see Fig. 9). It has been described in detail in a paper by Edward Schildhauer, M. Am. Soc. C. E.[9]

This machine, under constant motor speed, imparts an ideal motion to the gate, that is, slow speed at the ends of travel and high speed at the mid-point. The force required mainly depends upon the vertical area which is submerged during operation as the movement causes a difference in water level on the

[9] *Transactions*, Inter. Eng. Cong., 1915, Vol. II, Paper No. 20, p. 256.

two sides of the gate, which is very marked near the recess and mitered position and during acceleration. As the depth of submergence of the Illinois Waterway gates in no case exceeds 16 ft. when operated, or less than one-half the submergence of the Panama gates, the forces are much less and the machinery is lightened accordingly. Several other changes have been made in the design, among which may be mentioned the omission of the spring-center spur gear to reduce the starting torque of the motor. Experience has shown that the value of the spring center is not commensurate with its cost.

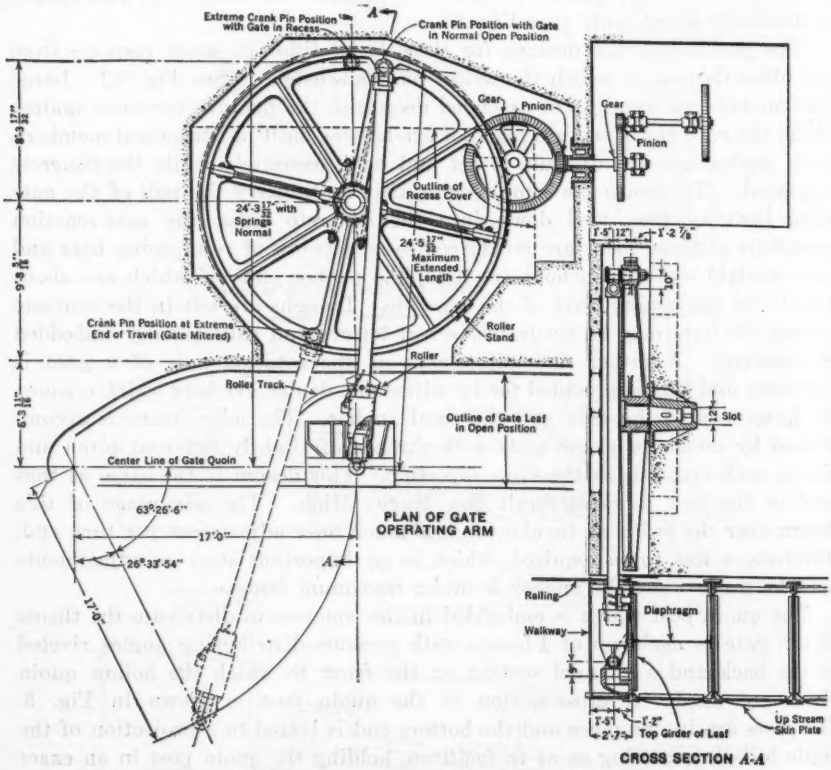

FIG. 9.—PLAN OF STRUT ARM AND BULL WHEEL FOR OPERATING MITER-GATES, ILLINOIS WATERWAY

LIFT-GATES AND OPERATING MACHINERY

At Lockport there exists a small modern lock built by the Sanitary District of Chicago to connect the Drainage Canal with the old Illinois and Michigan Canal. The necessity of retaining this lock so restricted the remaining available lock site in width that, adjoining the small lock, there was not sufficient area for the mass of concrete required for the thrust of a mitering gate. Hence, for these upper gates, a vertical lift, horizontal, truss-type of gate was adopted (see Fig. 10).

Two gates similar in construction and operation have been installed. One acts as an emergency gate and will be used only if it becomes necessary to remove the other for repairs. The structure consists of a movable gate, counterweighted, with a tower at each end, which is connected at the top by a service bridge placed at a clear elevation of 50 ft. above the high-water level. These gates have been described previously in a paper by the writer.[10]

The lock chamber is 110 ft. wide, and is provided at the upper end with recesses in the side walls about 5 ft. wide and 14 ft. long, that extend the

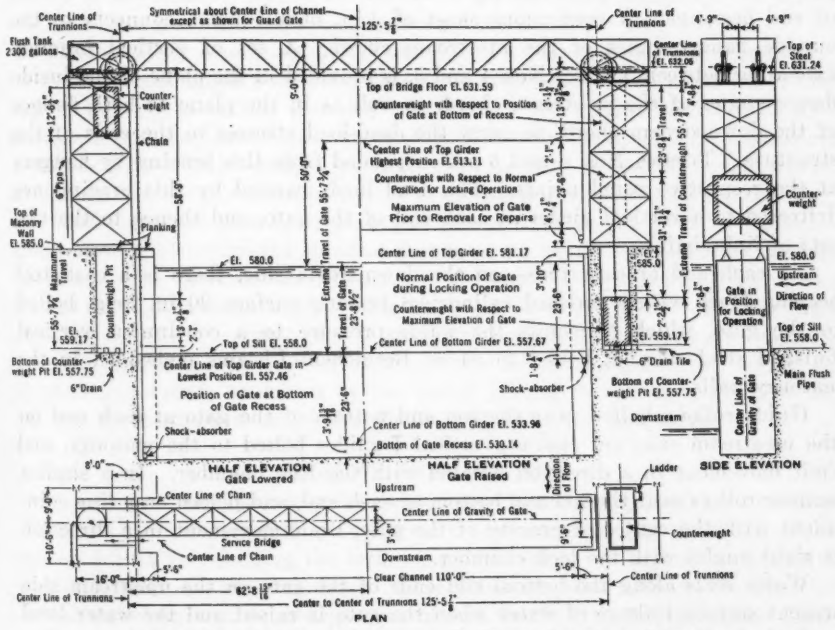
Center Line of Trunnions
Symmetrical about Center Line of Channel except as shown for Guard Gate
125'-5 7/8"
Flush Tank 2300 gallons
Counterweight
Chain
6" Pipe
Top of Masonry Wall El. 585.0
Planking
Top of Bridge Floor El. 631.59
Counterweight with Respect to Position of Gate at Bottom of Recess
Center Line of Top Girder Highest Position El. 613.11
Counterweight with Respect to Normal Position for Locking Operation
Maximum Elevation of Gate Prior to Removal for Repairs
Center Line of Top Girder El. 581.17
El. 580.0
Normal Position of Gate during Locking Operation
Counterweight with Respect to Maximum Elevation of Gate
Top of Sill El. 558.0
Center Line of Bottom Girder El. 557.67
Center Line of Top Girder Gate in Lowest Position El. 557.46
Position of Gate at Bottom of Gate Recess
Shock-absorber
Center Line of Bottom Girder El. 533.96
Bottom of Gate Recess El. 530.14
Bottom of Counterweight Pit El. 557.75
6" Drain
Counterweight Pit
HALF ELEVATION Gate Lowered
HALF ELEVATION Gate Raised
Center Line of Trunnions El. 632.06
Top of Steel El. 631.24
Counterweight
Gate in Position for Locking Operation
Upstream
Direction of Flow
Main Flush Pipe
Downstream
Center Line of Gravity of Gate
SIDE ELEVATION
Ladder
Center Line of Chain
Service Bridge
Clear Channel 110'-0"
Center Line of Trunnions
Center to Center of Trunnions 125'-5 7/8"
PLAN

FIG. 10.—LIFT-GATE OPERATING DIAGRAM, LOCKPORT LOCK

full depth of the lock, and the gate operates within these recesses. The vertical travel is 23 ft. 8½ in. and is accomplished in 2 min. by a 50-h.p. electric motor, placed at the center of the service bridge, and acting through the necessary reductions on internal gears integral with the sprocket drums. Two link chains at each end of the gate pass over these sprocket drums at the top of the towers and connect to a steel-skeleton counterweight box, filled with sufficient concrete to balance the weight of the gate when immersed.

The lift-gate consists of five horizontal trusses, hereinafter designated as Trusses Nos. 1, 2, 3, 4, and 5, and placed one below the other, Truss No. 1 being uppermost. Trusses Nos. 1, 2, and 3 are spaced 6 ft. 8 in., center to center; Trusses Nos. 3, 4, and 5 are spaced 5 ft. 1 in., center to center. Each truss is of the ordinary Pratt type, with a single web system. The web members and posts are composed of angles, and the down-stream or

[10] *Engineering News-Record*, Vol. I, 1926, p. 722.

tension chord of 15-in. channels is reinforced with 12-in. side-plates where necessary. The up-stream or compression chord is built up of four angles spaced 20 in. back to back, two 19 by $\frac{13}{16}$-in. web-plates, a 13-in. cover-plate outside the skin-plates, and reinforcing side plates where required. In addition to the direct compression, this chord was designed to carry the bending moments resulting from the uniformly applied water-pressure load transferred to it from the skin-plates.

Each truss is 11 ft. 9 in., center to center of chords, and consists of ten panels of 11 ft. 9 in. each, making a distance of 117 ft. 6 in., center to center of end bearings. A continuous sheet of $\frac{1}{4}$-in. buckle-plates connects to the outside flange angles of the up-stream chord. A set of vertical bracing, extending between Trusses Nos. 1 and 3, is provided in the plane of the inside flange angles of the up-stream chord, as well as in the plane of both flanges of the down-stream chord, to carry the dead-load stresses to the ends of the structure. Trusses Nos. 4 and 5 are suspended from this bracing by hangers at the respective panel points. The dead loads carried by this bracing are delivered to a vertical girder at each end of the gate, and thence to the two sets of link chains.

At each end of each truss, on the down-tream side, there is a cast-steel bearing block with a finished cylindrical bearing surface 20 in. long, bolted to the gate, which transmits the water pressure to a continuous vertical finished steel casting, bolted to 24-in. Bethlehem I-beams embedded in the masonry walls.

Guide rollers, bolted near the top and bottom of the gate at each end on the up-stream side, act against vertical T-guides bolted to the masonry, and limit movement in a direction parallel with the lock chamber. In a similar manner rollers near the top and bottom at each end, and placed on a line coincident with the center of gravity of the gate, limit movement in a direction at right angles with the lock chamber.

Water seals along the bottom and ends of the gate on the up-stream side prevent serious leakage of water when the gate is raised and the water level in the lock chamber is lowered. The bottom seal consists of a continuous line of $4\frac{1}{2}$-in. standard wrought-iron pipe, connected together by screw plugs and resting on a horizontal curb angle just below the top of the breast wall. This pipe terminates at each end in the side of a casting, resting on, but not bolted to the masonry, and the top of this casting serves as a support for another $4\frac{1}{2}$-in. pipe forming the vertical seal. This vertical pipe also terminates in a casting bolted to the masonry at the upper end. Screw plugs are provided at each end of the vertical pipe and enter recesses in the castings. These recesses allow the necessary movement for the pipe to come in contact with the gate. The horizontal pipe forms a contact between the curb angle on the masonry and the cover-plate on Truss No. 5. Rivets coming in contact with all pipes are countersunk. A series of T-headed bolts, spaced 3 ft. 6 in., center to center, throughout the length of the horizontal pipe and anchored to the masonry (the heads being inserted through slotted holes in the pipe and then turned 90°) prevent this pipe from falling out of position when the gate is raised for repairs. At the same time this arrangement allows freedom of

movement for the pipe to assume a curve corresponding to the deflection of the gate under maximum pressure. The vertical seal acts in a similar manner to the horizontal, forming a contact between a vertical curb angle embedded in the masonry and vertical plates riveted to the ends of the gate.

Owing to the great quantity of sludge in the drainage-canal water a flushing system for the bottom seal and recess is necessary. A flushing pipe is placed in the breast-wall masonry just back of the recess with smaller pipes leading from it into the recess at frequent intervals. This pipe is connected with a tank placed on one of the towers of the operating bridge so that water can be discharged into the recess under a 60-ft. head.

The operating machinery of the lift-gate is at the center of a tower bridge, 50 ft. above the upper pool water level. Sprocket drums are mounted on the top of the towers and by specially constructed link chains suspend the lift-gates and the counterweights at each corner, equilibrium being established when the gate is in its middle position (see Fig. 10). A 50-h.p. electric motor, at 600 rev. per min., was agreed upon as the basis of calculations for the moving machinery, the time of operation to be 2 min. for a lift of 25 ft. Internal gears cast *en bloc* with the sprocket drums are meshed with operating pinions mounted on a shaft between them and are revolved by a large gear at the center through a gear train ending in a set of miter gears. A line shaft running the entire length of the tower bridge rigidly connects the two systems of gears and drums at each end, a small gear being coupled at the center. A short counter-shaft is driven by the motor pinion at one end, while at the other a sprocket-wheel and roller chain facilitate hand operation.

Lockport Emergency Dam

A lock-gate may be damaged or wrecked by boats going in either direction. The most common accident occurs from a misunderstanding of signals on an upbound boat approaching the lock when it is filled. In case of accidents to the lock-gates at Starved Rock, Marseilles, Dresden Island, or Brandon Road, the water can be drawn down almost to the level of the upper miter-sill by opening the movable dams.

At the summit level at Lockport it is not possible to draw the pool down in this manner. An emergency gate has been designed therefore to be installed up-stream from the upper lock-gate. In case any of the gates at this lock are carried away the emergency dam can be raised and the flow of water reduced.

An outline of this emergency dam is shown in Fig. 11. It consists of an immense steel shutter swiveling on a horizontal axis operated by valves in manholes in the side walls. Ordinarily, this shutter rests in a horizontal position in a recess in the floor. Beneath the up-stream half of this shutter there is a recess in which this part of the gate swings. This recess can be connected with either the upper or lower pool by manipulating the proper valves in the side walls. When the recess is connected with the upper pool the pressure on both sides of this half of the shutter is equalized and, as the other leg of the shutter is longer, it will maintain a horizontal position. When it is desired to raise the shutter to an upright position the valves connecting the up-stream recess to the upper pool are closed and those connecting it

to the lower pool are opened. The pressure from the upper pool on the up-stream leg of the shutter, being then unbalanced, will force this part of the shutter down.

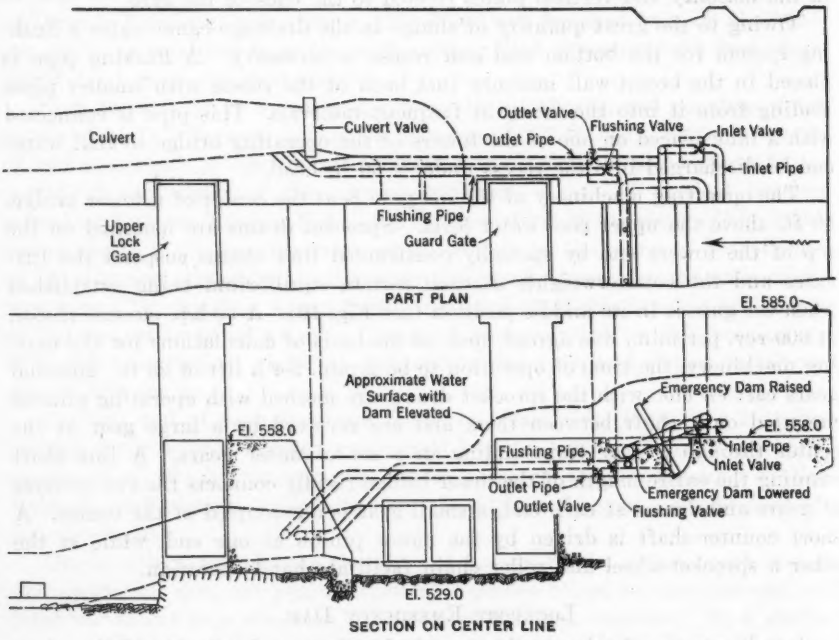

FIG. 11.—PLAN AND PROFILE, EMERGENCY DAM, AT LOCKPORT, ILL.

It may be noted from the diagram that a large pipe has been provided in each side wall connecting the upper pool with the recess under the down-stream leg of the gate. This is done to supply water and equalize the pressure on the face and back of the gate when it begins to rise and prevents the tendency for a partial vacuum in this recess to hold the shutter down. There is a pipe in the concrete beneath the recess, below the down-stream half of the shutter. At intervals there are smaller pipes, connecting the recess with this pipe, which blow out sludge and other deposits that may form in the recess from time to time. This can be done by connecting the pipe with the lower pool. It is very necessary on account of the large quantity of sludge carried by the Sanitary Canal.

When the dam has been lifted, the flow is cut down sufficiently for the water surface to take the elevations shown in Fig. 11. As all the water will then be going through the culverts the upper lock-gate or emergency gate, whichever is uninjured, can be raised. After this has been done and before the valves are closed, the emergency dam can be lowered by closing the valve connecting the space below the up-stream leg of the shutter to the lower pool and opening the one connecting it to the upper pool. The dam will then revolve into the recess in the floor.

Side-Wall Analysis

As the culverts in the locks are circular, the stress in the wall at the culvert cannot be found by the method used when they are rectangular. It is believed in this case that the most accurate method of determining the stresses in the masonry around the culvert is that of the "Principle of Least Work."

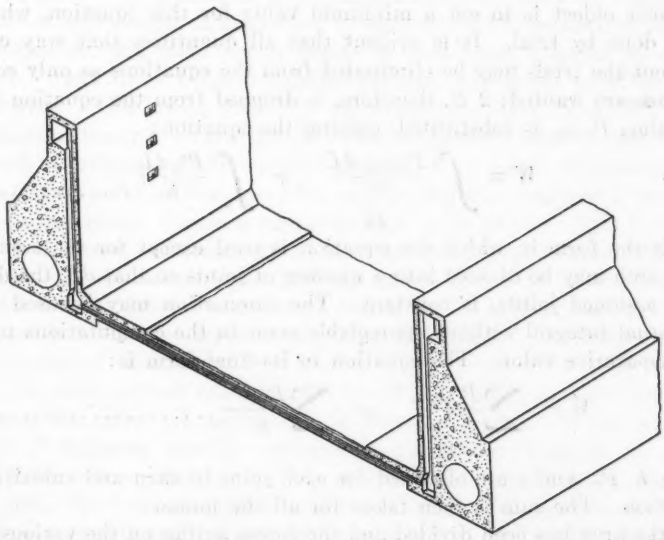

FIG. 12.—ISOMETRIC VIEW OF SECTION OF LOCK, ILLINOIS WATERWAY

In this analysis the assumption is made that the arch will do the least amount of work necessary to support the load upon it. Hence, if an expression can be found for the work done by each pressure line, the pressure line giving the least amount of work is the true one, and the stresses in the joints can then be computed.

The formula for the work of resistance in an arch (see Fig. 12) is:

$$W = \int_0^l \frac{M^2\, dL}{2\, EI} + \int_0^l \frac{P^2{}_n\, dL}{2\, EA_n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which,

W = total work of resistance in the structure.
M = bending moment at any assumed joint.
P_n = total pressure normal to section at any assumed joint.
L = distance from support measured along axis.
dL = an infinitesimally small distance along the axis of the arch.
l = length of arch measured along axis.
E = modulus of elasticity of material of arch.
A_n = area of section at any assumed joint where P_n acts.
I = moment of inertia of section of any assumed joint where P_n acts.
e_n = distance of point of application of P_n from center of section at any assumed joint.

Substituting h for the thickness of the arch at any joint and making the length of the arch considered equal to $b = 1$ ft., $A_n = hb = h$. The equation then becomes:

$$W = \int_0^1 \frac{M^2 dL}{2E\frac{h^3}{12}} + \int_0^1 \frac{P^2_n dL}{2Eh} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The next object is to get a minimum value for this equation, which can only be done by trial. It is evident that all quantities that stay constant throughout the trials may be eliminated from the equations as only comparative values are wanted; 2 E, therefore, is dropped from the equation and for M its value, $P_n e_n$, is substituted, making the equation:

$$W = \int_0^1 \frac{P^2_n e^2_n dL}{\frac{h^3}{12}} + \int_0^1 \frac{P^2_n dL}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This is the form in which the equation is used except for the assumption that the arch may be divided into a number of joints so that dL, the distance between assumed joints, is constant. The summation may be used instead of the actual integral without appreciable error in the computations to determine comparative value. The equation in its final form is:

$$W = \sum \frac{P^2_n e^2_n}{\frac{h^3}{12}} + \sum \frac{P^2_n}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which, h, P_n, and e are obtained for each joint in turn and substituted in the equation. The sum is then taken for all the joints.

After the arch has been divided and the forces acting on the various blocks have been found, a force polygon can be drawn and the resultants obtained from it in direction and magnitude. The point of application of each resultant can then be found by taking the sum of the moments of all the forces acting on one-half the arch about any point and dividing this moment by the resultant. This will give the lever arm of the resultant about the center of moments, and the resultant can then be drawn in position on the arch.

There are three unknown quantities to be obtained, the horizontal and vertical components and the point of application of the crown thrust, and all are determined very easily by this process. Any point on the springing line and crown joint may be taken and the horizontal crown thrust is obtained that will pass a pressure line through the point on the crown joint and the springing line. This is accomplished by taking moments of all the forces acting on that half of the arch about the point on the springing line and dividing the sum by the distance to the line of action of the crown thrust. This thrust can then be applied at various points on the crown joint, the pressure line can be drawn, and the work computed for each.

It is generally desirable to apply the thrust at about five points on the crown joint at equal distances apart. Then it is only necessary to draw the two outer pressure lines because the distance between these lines can be measured at each joint and divided by four in getting the values of e_n for

each line. After the work has been obtained for the set of pressure lines for this crown thrust, assume a thrust a little larger or smaller, pass pressure lines through the same points on the crown joint and through a minimum, and compute as before.

After computations have been made for five or six different crown thrusts the value of the work for each thrust at each different point of application should be plotted on a cross-section diagram with the work as ordinates and the point of application at the crown as abscissas. By connecting the points thus plotted for the same crown thrust in each case, a series of curves is obtained. Next, draw a curve passing through the lowest point on each of these curves. The lowest point on the resulting curve gives the point of application of the true crown thrust. Then on another cross-section diagram, with work as ordinates and horizontal component of crown thrust as abscissas, plot the lowest point on the curve for each of the trial horizontal components of the crown thrust. Draw a curve through these points. The lowest point will give the true horizontal component of the crown thrust. This leaves only the true vertical component of the crown thrust to be obtained.

As the true horizontal component and point of application of the crown thrust are now known, try several different vertical components using the same horizontal component and point of application in each case. Of course, for each vertical component it is only necessary to draw one pressure line since the point of application is already known. When a sufficient number of different vertical components have been tried in an effort to obtain a minimum, the results are plotted on a cross-section diagram as before, with the work as ordinates and the vertical components as abscissas. Passing a curve through these points and taking the lowest point on that curve, the true vertical component is obtained. Having the true vertical and horizontal components and point of application, the true pressure line may be drawn and stresses in the various joints computed.

The wall of the Marseilles Lock (see Fig. 13) is being built up to the level of the shoulder after which a wait occurs until the concrete sets sufficiently to erect the forms for the back of the wall upon it. This results in a well-defined joint on this line. The wall above this line is designed so that for the most adverse conditions possible there will be no tension at the heel. The stress diagram for this joint is then drawn as shown in Fig. 13(*a*) and the upper wall is assumed to be replaced by this pressure on the top of the arch. The arch is then divided into sections by imaginary joints and by the forces found acting on each of these blocks. A force diagram is then drawn, and the resultants acting on the two halves are obtained in direction, magnitude, and point of application, as previously explained. With the moments of all the forces on each half of the arch determined, it is very easy to obtain the horizontal component of crown thrust to pass a pressure line through any point on the springing line. It will be noticed from Fig. 13 that the trial crown thrusts used are not horizontal, but very much inclined. It is not necessary to use a horizontal crown thrust, but in the first set of computations to find the true horizontal component and point of application, the vertical component must be the same for each trial.

A trial computation for a horizontal component of 12 000 lb. is shown by Fig. 13 and the computations are given in Table 1. In drawing the pressure lines on the arch it is well first to prolong the crown thrusts at the extremes

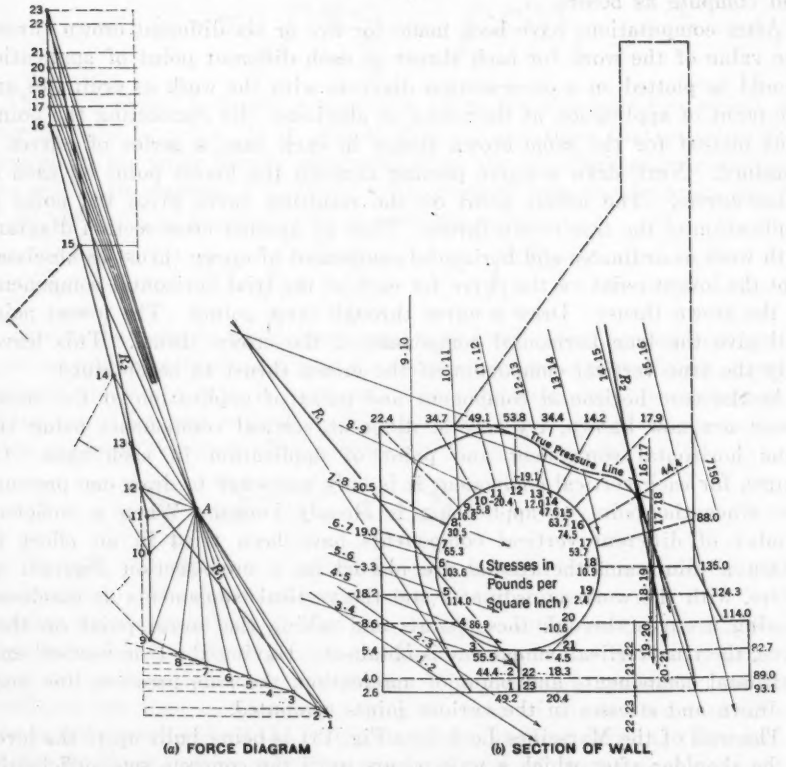

FIG. 13.—ANALYSIS OF STRESSES IN CULVERT WALL, MARSEILLES LOCK

out to the resultants and then draw lines from their intersections on the resultants, parallel with the line in the force diagram representing the resultant of the crown thrust and all the forces on one-half the arch. These lines are $A1$ and $A23$ in the force diagram, Fig. 13(a). This will give points on the springing line to which the pressure lines drawn from joint to joint must come if correct. Only the two outside pressure lines are drawn although there are five used in the computation. The intermediate values of e_n can be obtained very easily, as they are all an equal distance apart. The values of P_n are the same on each joint for all the lines. It will be noticed in Table 1 that they are divided by 1 000, so that the numbers in the computations will not be too large.

After completing this computation another value of crown thrust is taken and a similar table is computed for it. Four different values of crown thrust were used in this case, the horizontal components being 13 000, 12 000, 11 000, and 9 000 lb. The values of work for each of these crown thrusts for each

TABLE 1.—COMPUTATION OF STRESSES IN WALL OF MARSEILLES LOCK FOR A HORIZONTAL COMPONENT OF 12 000 POUNDS

Joint	h, in inches	$\frac{h^3}{12}$	$\frac{P_n}{1000}$ (kips)	$\left(\frac{P_n}{1000}\right)^2$	Column (5) / Column (3)	Column (5) / Column (2) $\left(\frac{P_n}{1000}\right)^2 \div h$	e_{-2} (feet)	e_{-2}^2 (square feet)	Column (6) × Column (9)	e_{-1} (feet)	e_{-1}^2 (square feet)	Column (6) × Column (12)	e_0 (feet)	e_0^2 (square feet)	Column (6) × Column (15)	e_1 (feet)	e_1^2 (square feet)	Column (6) × Column (18)	e_2 (feet)	e_2^2 (square feet)	Column (6) × Column (21)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
1	10.0	83.3	38.1	1452	17.5	145.2	1.70	2.89	50.5	1.38	1.90	33.2	1.07	1.14	11.9	0.75	0.56	9.8	0.43	0.19	3.2
2	10.0	83.3	36.6	1340	16.1	134.0	1.22	1.49	24.0	0.89	0.70	11.3	0.55	0.30	4.9	0.22	0.05	0.8	0.12	0.10	0.2
3	7.5	35.2	33.6	1129	32.1	150.6	1.52	2.31	74.3	1.15	1.32	42.4	0.78	0.61	19.5	0.41	0.17	5.4	0.04	0.00	0.0
4	5.6	14.6	32.3	1043	71.3	186.5	1.67	2.79	199.0	1.30	1.69	120.4	0.93	0.87	61.7	0.56	0.31	22.4	0.19	0.04	2.6
5	4.4	6.9	30.7	942	137.5	216.5	1.77	3.13	430.0	1.37	1.88	258.0	0.97	0.94	120.4	0.58	0.34	46.2	0.18	0.03	4.4
6	4.0	5.3	29.6	876	164.5	219.0	1.63	2.66	438.0	1.23	1.51	249.0	0.83	0.69	113.3	0.43	0.19	30.5	0.03	0.00	0.2
7	4.4	6.9	27.7	767	112.0	176.3	1.50	2.25	270.0	1.07	1.14	137.0	0.64	0.41	49.2	0.21	0.04	5.3	0.22	0.05	5.8
8	5.5	13.9	25.8	666	48.1	121.0	1.41	1.99	65.7	0.94	0.88	42.3	0.47	0.22	10.6	0.00	0.00	0.0	0.47	0.22	10.6
9	8.1	44.3	24.6	605	13.6	74.8	1.49	2.22	31.0	0.99	0.98	13.3	0.49	0.24	3.3	0.00	0.00	0.0	0.50	0.25	3.4
10	5.6	14.2	13.6	185	13.0	33.7	1.84	3.39	44.1	0.96	0.92	18.8	0.06	0.00	0.1	1.14	1.30	23.4	2.22	4.93	88.8
11	4.4	6.9	11.1	123	18.0	28.3	2.10	4.41	79.5	1.02	1.04	27.1	0.00	0.00	0.0	1.00	1.00	27.1	2.00	4.00	108.4
12	4.0	5.3	12.0	144	27.1	36.0	2.00	4.00	108.4	1.00	1.00	27.1	0.00	0.00	0.0	1.00	1.00	27.1	2.00	4.00	108.4
13	4.4	6.9	16.2	262	38.3	60.3	1.92	3.69	141.5	1.18	1.39	53.3	0.44	0.19	7.4	0.30	0.09	3.5	1.04	1.08	41.4
14	5.6	14.2	26.1	681	47.9	122.6	1.75	3.06	146.5	1.30	1.69	81.0	0.85	0.72	34.6	0.40	0.16	7.7	0.85	0.72	34.6
15	8.2	45.9	48.7	2372	51.8	289.0	1.41	1.99	103.0	1.17	1.37	71.0	0.93	0.87	44.8	0.69	0.48	24.6	0.45	0.20	10.6
16	9.0	60.7	72.6	5271	86.8	585.0	0.68	0.46	40.1	1.52	0.27	23.4	0.36	0.13	11.3	0.20	0.04	3.5	0.04	0.00	0.2
17	7.5	34.4	75.8	5746	167.0	772.0	0.05	0.00	0.5	1.23	0.05	8.9	0.41	0.17	28.1	0.59	0.39	58.1	0.77	0.59	97.4
18	7.1	28.6	73.3	5373	187.6	762.5	0.90	0.81	152.0	1.07	1.14	214.0	1.24	1.54	289.0	1.41	1.99	374.0	1.58	2.50	469.0
19	8.4	33.8	75.5	5700	168.6	683.0	1.28	1.64	276.5	1.43	2.04	344.0	1.58	2.50	421.0	1.73	2.99	504.0	1.88	3.53	595.0
20	10.6	72.6	77.9	6068	83.6	575.0	2.06	4.24	354.0	1.22	4.93	412.0	2.38	5.66	473.0	2.54	6.45	539.0	2.70	7.23	609.0
21	14.5	254.0	80.6	6496	25.5	448.0	2.66	7.07	180.0	2.81	7.90	202.0	2.96	8.76	223.5	3.12	9.73	248.2	3.28	10.76	274.0
22	17.0	410.0	85.7	7344	17.9	432.0	4.28	18.32	328.0	4.43	19.62	352.0	4.58	20.98	376.0	4.73	22.37	400.0	4.88	23.81	427.0
23	17.0	410.0	88.3	7797	19.0	459.0	4.49	20.16	384.0	4.63	21.44	480.0	4.77	22.75	433.0	4.91	24.11	459.0	5.05	25.5	486.0
Summation	.....	.....	.....	.....	.....	6710.8	.....	.....	3 950.6	.....	.....	3 134.4	.....	.....	2 745.7	.....	.....	2 800.8	.....	.....	3 308.5
	.....	.....	.....	.....	.....	.....	.....	.....	6 710.8	.....	.....	6 710.8	.....	.....	6 710.8	.....	.....	6 710.8	.....	.....	6 710.8
	.....	.....	.....	.....	.....	.....	.....	.....	10 661.4	.....	.....	9 845.2	.....	.....	9 456.5	.....	.....	9 511.6	.....	.....	10 019.3

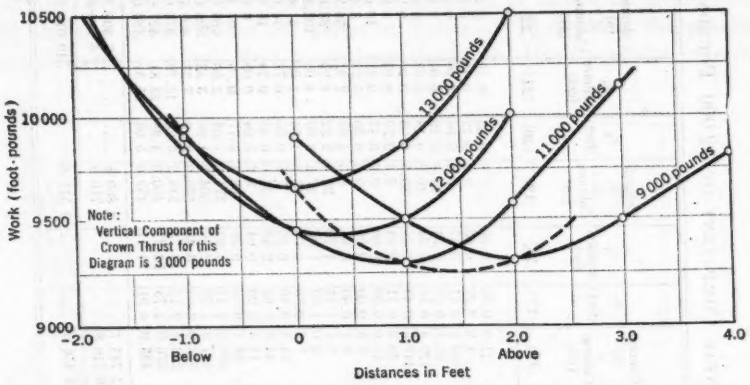

FIG. 14.—DISTANCE OF POINT OF APPLICATION FROM CENTER OF CROWN JOINT

point of application were then plotted as shown in Fig. 14 and a curve connecting the points for each crown thrust was drawn. A curve was then drawn through the lowest point on each of these curves and thus a value of 1.4 ft. above the center was obtained for the point of application for the true crown thrust. The lowest points of each of the curves for different crown thrusts were then plotted as shown in Fig. 15(*a*) and a value of 10 000 lb. was obtained for the horizontal component of the true crown thrust.

Using this horizontal component of crown thrust and the point of application from Fig. 14, three computations were made, assuming vertical components of 2 000, 3 000 and 5 000 lb. The table for this computation is the

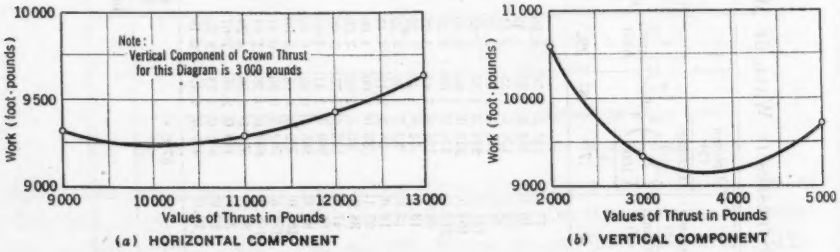

FIG. 15.—COMPONENTS OF THRUST AT THE CROWN

same as that used for obtaining the horizontal component, except that for each vertical component assumed only one pressure line is drawn, as the point of application is known. The values obtained for the different vertical components are then plotted as illustrated in Fig. 15(*b*), and a curve is drawn through these points. The lowest point on this curve gives a value of 3 700 lb. as the true vertical component.

Having now obtained the true vertical and horizontal components and point of application of the crown thrust it is a simple matter to draw the true pressure line and compute the stresses in the various joints. The pressure line and stresses obtained for the section in question are shown in Fig. 13.

It is not claimed that this method will give exact results, but it is believed to give results nearer the truth in a case of this kind than any other method. The computation would be rigidly true if the arch were divided so that $\frac{dL}{l}$ is constant for the first term of the formula. Perform this part of the computation, then divide the arch so that $\frac{dL}{A_c}$ is constant and use this division for the second term of Equation (2).

In designing the arched conduits through the dams for the Miami Conservancy District, at Dayton, Ohio, this was done as a check for one of the arches after the true pressure line had been obtained by the method described herein. It was found that the pressure lines obtained by the two methods were practically identical; the crown thrust differed only 0.17% and the point of application was practically the same.

Taintor Gate Dams and Operating Machinery

The dams at all the sites except Brandon Road consist of a concrete weir of moderate height, with piers at certain intervals. A movable gate that may be opened during flood periods is placed between each two piers. At one end of each of the dams the height of the gate is different from the others, similar to the sector gate at Lockport. This is for the purpose of taking care of the ice (see Section *B–B*, Fig. 16).

Three types of gates were studied for the movable parts of these dams. The first consisted of a shutter, hinged along the up-stream edge, which is at the level of the crest of the permanent masonry portion of the dam. At the two down-stream corners chains from heavy counterweights are attached, the counterweights sliding along the backs of the piers.

The second consisted of the well-known "Stoney gate" type, consisting of a large, counterweighted shutter traveling vertically in grooves. Instead of having a train of rollers for each end bearing traveling with the gate at half speed, as is the custom in this type, fixed wheels with roller bearings were substituted.

The third type is a "Taintor gate" and consists of a part of a cylindrical surface braced to and revolving about a horizontal axis. It is counterweighted so that the operating machinery will have only friction to overcome. This gate can be counterweighted accurately much more cheaply and simply than either of the others. The operating machinery is carried by a light steel footbridge spanning from pier to pier.

The first type was quickly discarded because of the great difficulty of counterweighting due to the extreme variation of the load for different positions of the gate. This would make the cost of operating machinery and power very heavy.

In studying the second and third types the cost was found to be about the same, but there were several disadvantages of the second, or "Stoney" type. A heavy operating car with powerful machinery would have to be provided for installing the gates and removing them when necessary, and a heavy

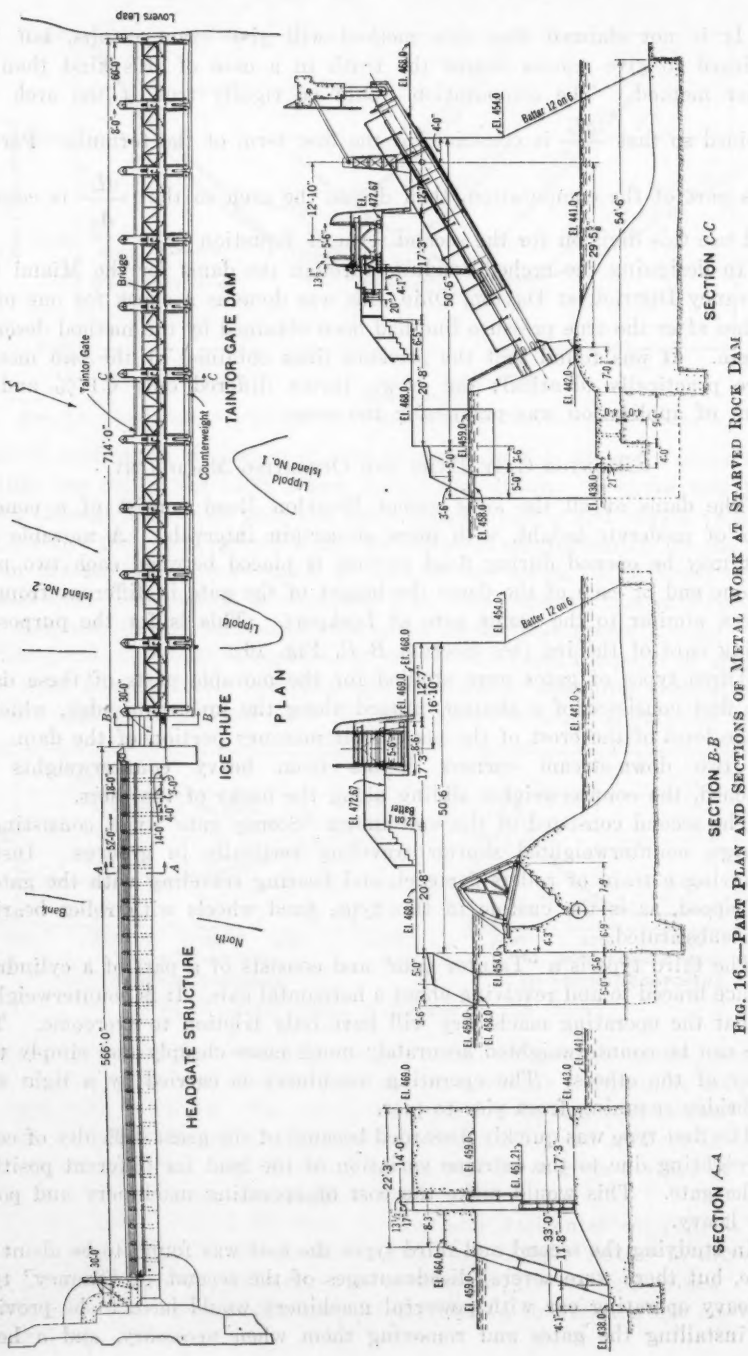

FIG. 16.—PART PLAN AND SECTIONS OF METAL WORK AT STARVED ROCK DAM

overhead bridge would be necessary to carry this car and the suspended gate. This bridge would have to be on high piers with its floor far above the tops of the lock walls, thus necessitating a stairway at each end and making it difficult to install or take out either the operating car or the gates. To offset this cost, the piers in the second type were much lighter than those in the third type, the weir being the same in each. This saving in concrete practically balanced the extra cost of the operating car and heavy bridge, but on account of the difficulty in installing and operating the Stoney type the "Taintor" gate type was adopted. Fig. 16 shows the layout of metal work at Starved Rock Dam.

A method has been provided to close any one Taintor gate-opening so that a gate may be inspected, repaired, and, if necessary, taken out and replaced without loss of water. Notches are provided on each side of each pier at the water surface into which a steel coffer (the bottom of which is a very strong horizontal beam) can be floated. Steel needles are inserted with the lower ends resting in a groove in the concrete apron and the upper ends against the bottom girder of the coffer. After the needles are placed, cocks in the bottom of the coffer are opened, and the water is drained out. These cocks are provided so that the coffer can be made to float at the proper level to fit into the notches in the piers.

All Taintor gates are operated by machinery placed on the piers. An alternative hand and motor operation is provided as indicated in Section *C–C*, Fig. 16. A shaft operated by a worm-wheel at the pier extends to the center of the operating bridge. A pinion on the end of this shaft, at the center of the bridge, engages a spur gear on a shaft that extends the entire length of the bridge. On the ends of this shaft are pinions that engage in the operating struts attached to the end struts of the gates, which extend beyond the center of rotation and carry the counterweight. The purpose of this method of operation is to insure the same amount of torsion at both ends of the operating shaft and thus prevent twisting in the gate.

Adjustable Crest-Sector Dam at Each Site

This dam (Section *C–C*, Fig. 16) is built of structural steel and is a part of a cylinder rotating about a horizontal axis. The curved surface is faced up stream so that the pressure of the water is exerted through the center of rotation. The weir is 30 ft. long, with an adjustable crest of 7 ft. maximum variation. It is constructed to facilitate the movement of ice. The curved sheet of the sector has a radius of 10 ft. A weir tube controlling the water level under the dam can be manipulated to regulate the amount of pressure head within the movable dam. The hinge consists of steel castings bolted together in sections. The upper, being part of the gate, is held by four pins to the lower castings embedded in the masonry and anchored. The castings run the entire length of the dam. A seal is obtained by arranging a curved overlapping apron of the upper casting so that it is in rubbing contact with a movable strip of metal secured to the lower stationary casting by set screws. Steam pipes prevent ice from forming about the hinges. The sides of the movable structure are sealed by pipes pressed against steel plates attached

to the walls of the flanking masonry piers. The gate can be locked in the elevated position by bolts keyed to a shaft on the breast-wall which holds the gate in position, while the water pressure inside can be released. A suitable manhole gives access to the interior of the chamber for cleaning or repairs. This dam is like that of the Sanitary District of Chicago at Lockport.[11]

Bridges

Twelve highway and street bridges that crossed the river channel either had to be rebuilt entirely or a draw span had to be constructed. Five of these bridges are in the City of Joliet and the others at various points along the Waterway. The five in Joliet and one at Brandon Road are rolling-lift bascule structures of the Scherzer type. The others are ordinary fixed bridges built at a high level.

In addition, there are draw spans in five railroad bridges that cross the river. One of these located in Joliet was entirely rebuilt.

Water-Pipe Tunnel

The Illinois Waterway interfered with the water pipes that crossed on existing bridges. The law authorizing construction required the State to provide a means of conveying the water across the canal before the existing mains could be disturbed. As the bridges at Joliet are of the rolling-lift bascule type, it was impossible to place any water mains on them and, therefore, a tunnel in which water mains could be placed had to be constructed under the river. The Electric Light and Power Company, the Telephone Company, and the Gas Company, also desired to use the tunnel, so they each paid their proportional part of the cost of the structure.

The section of the tunnel is oval, the longer diameter being horizontal. It has a maximum height of 9 ft. and a maximum width of 10 ft. The shafts are the same size as the tunnel. Conduits for electric light and power wires are provided in the side wall of the tunnel and in shafts on one side, with a manhole at the top of each shaft. Similar conduits are provided on the other side for telephone wires, also with manholes at the top of each shaft. Two 16-in. water mains and three gas pipes occupy the open space of the tunnel and shafts.

The center line of the tunnel is about 38.5 ft. below the bed of the stream and is in first-class limestone. There is a working chamber at the head of each shaft. Ordinarily, the tunnel and shafts will be kept flooded to within a few feet of the working chamber floors as this makes it safe to place gas mains, and electric light, power, and telephone wires in the same chamber.

A sump is provided at the bottom of one of the shafts, in which a rotary pump is placed with a shaft running up to a motor in the working chamber and a discharge pipe running up to and entering an intercepting sewer in the wall along the side of the canal. When it is desired to inspect the tunnel this pump will clear the tunnel of water in about seven hours. A blower is provided in the same working chamber to force air down the shaft, through the tunnel and up the shaft on the far side of the canal, before any one enters.

[11] *Engineering News*, Vol. II, 1908, p. 512.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

BY MORRIS MASON,[1] JUN. AM. SOC. C. E.

SYNOPSIS

This paper deals specifically with the Kittitas Division of the Yakima Project, in the State of Washington, constructed by the United States Bureau of Reclamation. All irrigation projects undertaken by the Bureau are similar in character, although new ideas of design and new methods of construction are introduced in each new structure. Most of the structures on any one project are small in size, but generally the topography of the ground and the type of structure are such that the method of constructing each unit must be different from any other. There is a great variety in size and type of structures. The choice of a method of construction is placed in the hands of the contractors, subject to the approval of the engineers. In the case of small units, it is uneconomical to install elaborate and expensive set-ups of machinery.

It is the purpose of this paper to give the history and general description of the Kittitas Division of the Yakima Project as a whole, with an enumeration and description of the various structures, and the more common methods of construction. The large quantity of concrete involved (150 000 cu. yd.), varying in strength from mass concrete in the diversion dam to the extremely high strength concrete in the high-head siphons (all of which must be of high impermeability), made necessary a very strict supervision of the proportioning and mixing of the aggregate. The method by which a concrete of known and uniform quality was attained, will also be of interest to engineers engaged in this class of work.

NOTE.—Written discussion on this paper will be closed in January, 1932, *Proceedings*.

[1] Engr., Standard Oil Co. (Indiana), Wood River Refinery, Wood River, Ill.

History

The history of irrigation in the Kittitas Valley, situated in the central part of the State of Washington, dates back to before the appearance of white men in that region. As farming developed in a larger way and more land was put under cultivation, there was more demand for water. Private and community low-level canals were then put in to serve a few farms along the Yakima River and its tributaries. As early as 1889 active survey work was in progress for the construction of a high-line canal to serve the farms on the higher benches and to provide more water for the low farms. Actual construction was begun on some of these canals, but it resulted in failure because of lack of funds. Stretches of the canals are still in evidence. Some survey work was done by the Government on the High-Line Canal prior to 1910. The Kittitas Reclamation District entered into a contract with the United States for water from Government reservoirs in 1921. In 1923 a board of engineers appointed by the United States Bureau of Reclamation decided that the Kittitas Unit should be selected for first construction, four divisions of the Yakima Project being considered. These were:

1.—Kittitas Division, covering approximately 72 000 acres of irrigable land, near Ellensburg, Wash.
2.—Moxee Division, covering about 35 000 acres of irrigable land, just east of Yakima, Wash.
3.—Rosa Division, covering about 60 000 acres of irrigable land, above the present Main Sunnyside Canal and lying north of the Towns of Sunnyside and Grandview, Wash.
4.—Kennewick Division, covering about 35 000 acres of irrigable land, west of the Columbia River at its confluence with the Yakima River near the Town of Kennewick, Wash.

Surveys were begun in 1924. General authority for the construction of the Kittitas Division was contained in a contract dated December 19, 1925, between the United States and the Kittitas Reclamation District. The active construction period began in April, 1926. Prior to this time the feasibility of the project from an engineering and economic standpoint had been thoroughly investigated. Topographic and soil surveys of the entire area were made, and the value of each 40 acres, or less (according to the legal subdivision lines, and including improvements), had been determined by a Board of Appraisers appointed jointly by the Government and the Reclamation District.

General Description

The Kittitas Division of the Yakima Project includes the arable lands of the Kittitas Valley in the State of Washington, with the exception of the bottom-lands which have been irrigated from the Yakima River for many years. Figs. 1 and 2 show the part of the Kittitas Valley to be embraced in the District. The main body of the land includes ranges in elevation from 1 700 to 2 100 ft., and surrounds the older irrigated lands in the form of an ellipse, with the Yakima River as the major axis and the City of Ellensburg near the center. (See Fig. 2.)

The area included within the limits of the Division is approximately 96 000 acres. The estimated irrigable area is 72 000 acres, of which 58 250 acres are in private hands; 6 750 acres are owned by the Northern Pacific Railway Company and by its subsidiary, the Northwestern Improvement Company; 5 400 acres are public land; and 1 600 acres belong to the State. Two impounding reservoirs are in use in connection with the Lower Yakima Project, surrounding Yakima, Wash. These reservoirs are situated at Lake Keechlus and Lake Kachess. (See Fig. 1(*a*).) From these points, water is fed into and supplements the natural flow of the Yakima River. Diversion into the Kittitas Main Canal is affected at the foot of the Cascade Mountains through the construction of a dam north of Easton. The Main Canal is parallel to the Yakima River for approximately 25 miles. The water is then divided at the South Branch Head-Works (Fig. 1(*b*)). Most of the water is carried in the North Branch Canal about a mile to the Yakima River Crossing, from which point the canal extends around the north and east sides of the Kittitas Valley, about 36 miles to the Whipple Pumping Plant (Fig. 2). Here, a portion of the water is lifted to a higher level by a direct-connected turbine pump. The gravity canal, pump canal, and drop canal extend around Badger Pocket. These laterals end at the mouth of the Yakima River Canyon about 6 miles southeast of Ellensburg. At the South Branch Head-Works 250 sec-ft. of water is diverted from the Main Canal. From there, it is carried in the South Branch Canal down the south side of the Kittitas Valley about 14½ miles to Manastash Creek, from which point delivery is made as far as the Yakima River Canyon by a main lateral.

Personnel Organization

All work is done under the supervision of the Construction Engineer. The Office Engineer and the Principal Field Engineer have headquarters in the Project Office in Ellensburg. The location engineers, designers, draftsmen, bookkeepers, and clerks are under the supervision of these men. Minor designs, including the preparation of steel bending and cutting lists, are made in the Project Office, where, also, purchasing and material accounts are kept. Division engineers are in direct charge of construction, each having charge of a separate division of the Project. Each Division Engineer has an office engineer, clerk, field parties, and inspectors. There were four Division Offices in operation on the Project, but only three at any one time. The work on the First Division was completed in sufficient time for the engineer in charge of that office to direct the work in the office of the Fourth Division also. All monthly estimates for contract payments are made in the office of the Division Engineers.

Structures of major importance are designed in the office of the Chief Engineer in Denver, Colo., who has general supervision of all irrigation work. The Commissioner of the Bureau of Reclamation, in Washington, D. C., is in charge of all work done by the Bureau.

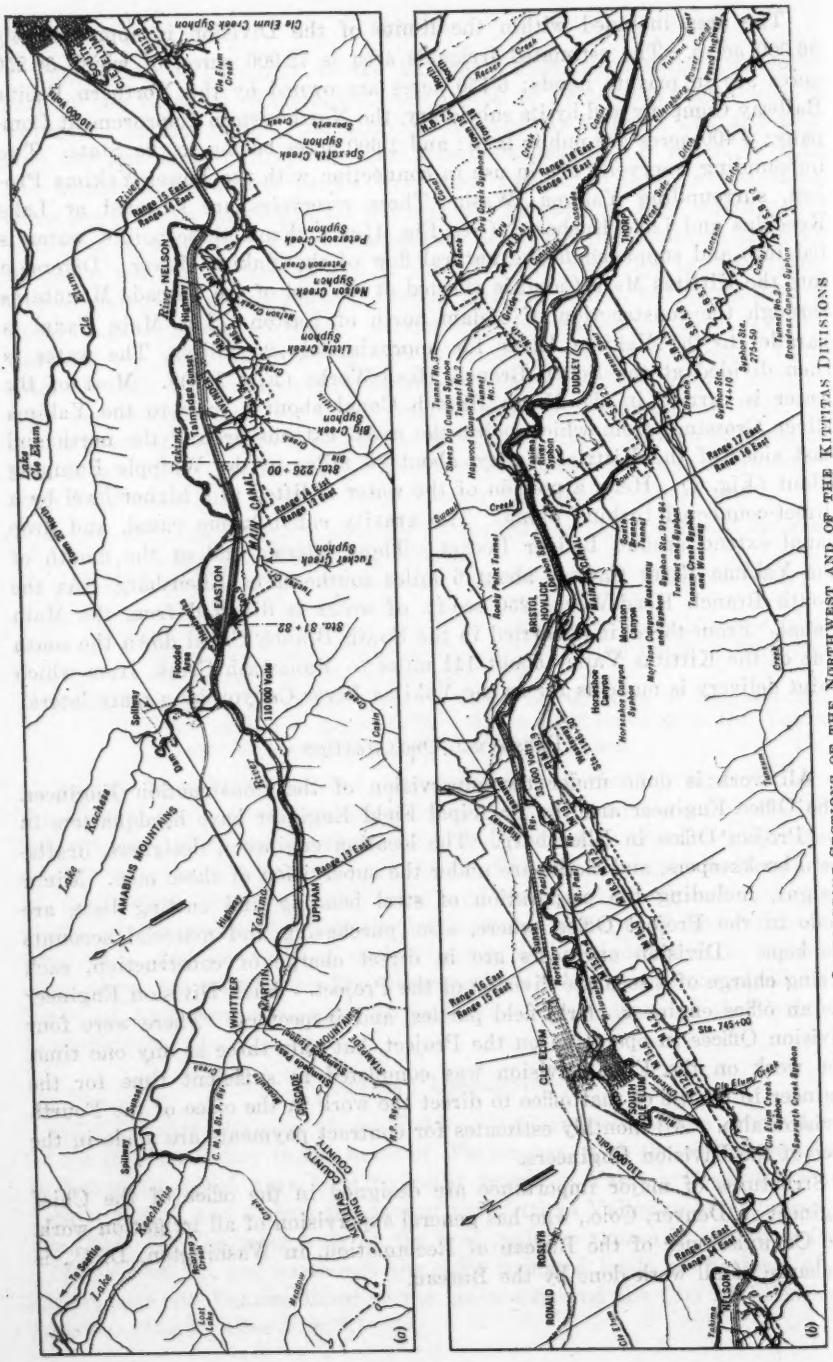

FIG. 1.—OVERLAPPING SECTIONS OF THE NORTHWEST AND OF THE KITTITAS DIVISIONS

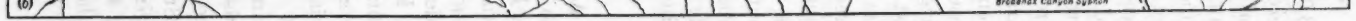

FIG. 1.—OVERLAPPING SECTIONS OF THE NORTHWEST AND OF THE KITTITAS DIVISIONS

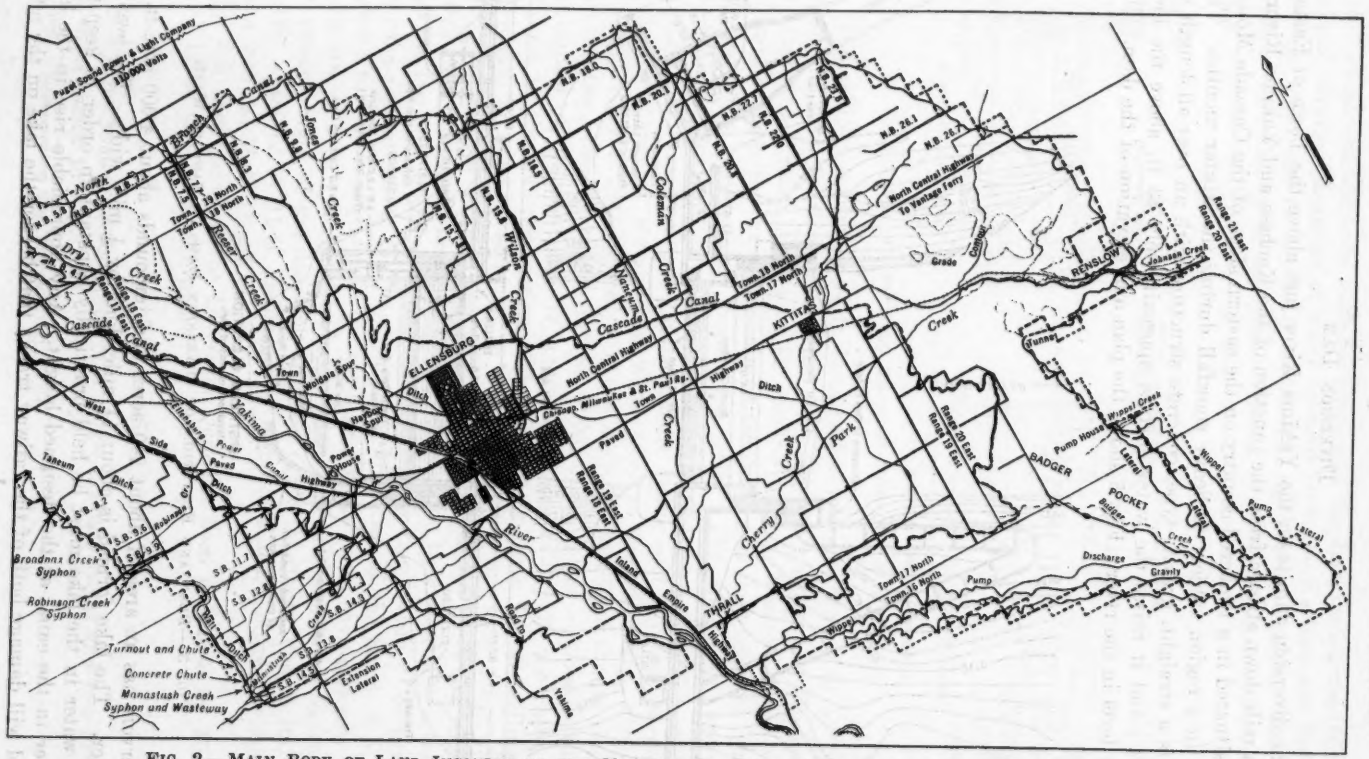

FIG. 2.—MAIN BODY OF LAND INCLUDED IN THE KITTITAS PROJECT; FARTHER EAST OF, AND OVERLAPPING, FIG. 1.

Diversion Dam

The diversion dam is on the Yakima River just above the Town of Easton and a mile down stream from the junction of the Kachess and Yakima Rivers. It is situated in a timbered country on the eastern slope of the Cascade Mountains in a region subject to heavy snowfall during the winter months. The dam is a straight, gravity-type, concrete structure with an over-all length of 248 ft.; and it raises the water surface approximately 43 ft. above the low-water level in the river. Fig. 3 shows the plan and elevation of the dam. The

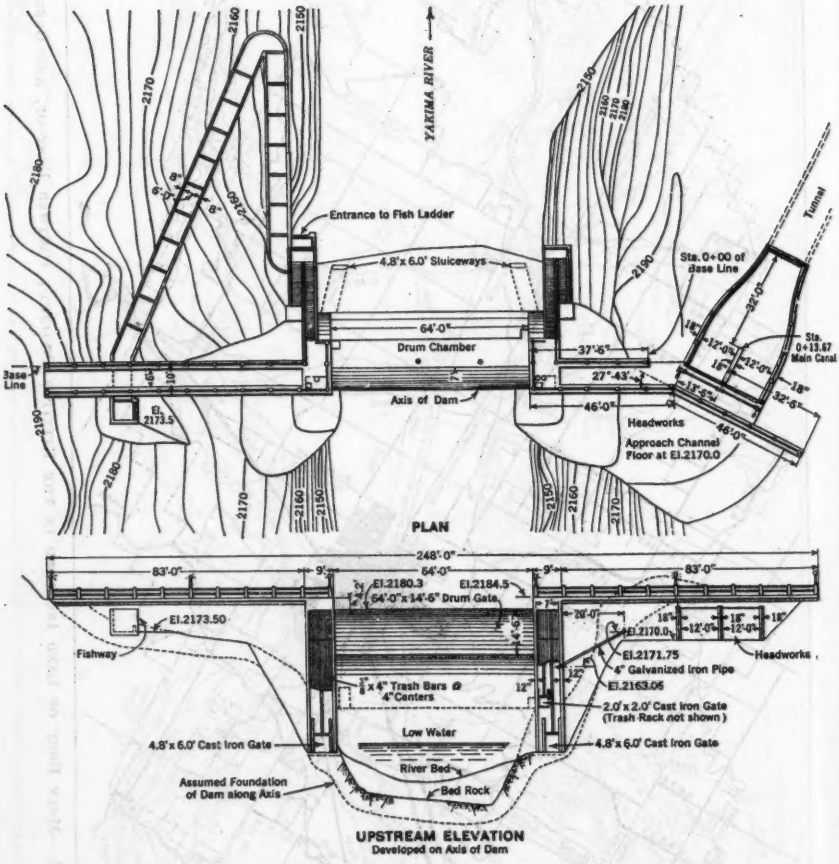

Fig. 3.—Plan and Elevation of Diversion Dam at Easton, Wash.

reservoir has an area of about 240 acres and impounds about 4 000 acre-ft. of water. The lake formed is about $1\frac{1}{2}$ miles long by $\frac{1}{2}$ mile wide. The level of the water in the lake must be held practically constant in order to keep the water in the canal at the designed height. Any appreciable rise in the lake level will damage both of the railway tracks that border the lake on the south

side. Provisions had to be made for passing by the dam, during the irrigation season, varying quantities of water released from the Kachess and Keechlus Storage Reservoirs for the lower divisions of the Yakima Project. The water level is controlled by an automatic floating, drum-type gate mounted on the ogee crest of the dam (see Fig. 4).

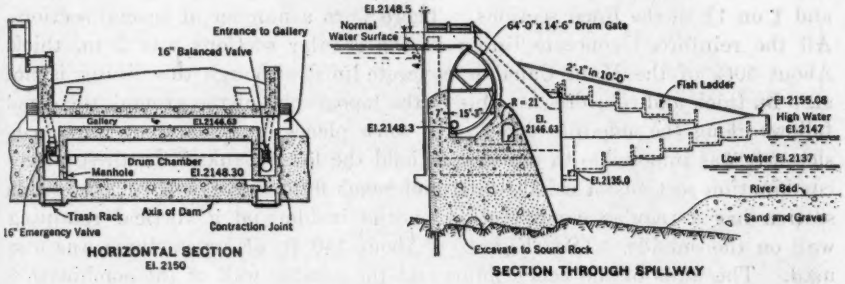

FIG. 4.—SECTIONS THROUGH DIVERSION DAM

The spillway section is 64 ft. long, and is provided with a 64 by 14½-ft. structural steel, floating drum-gate. The automatic control of the gate is accomplished through a float which operates a 16-in. balanced valve which, in turn, controls the water pressure in the drum-gate float chamber beneath the gate and raises and lowers the gate. The automatic operating mechanism holds the water level in the reservoir at a constant elevation of 2 180.3 ft., the drum-gate automatically rising and lowering to accommodate a change in elevation of water level in the reservoir. The spillway capacity with gate lowered is 13 000 sec-ft. while the maximum recorded flow of the river at this point is 6 000 sec-ft. The automatic control mechanism of the drum-gate is contained in the gallery in the right pier, adjacent to the spillway section. The emergency valve for controlling the drum gate by hand operation is in the left pier. Two 4.8 by 6.0-ft. sluice-gates are provided at the river level of the dam. These gates are used for sluicing out the sand which accumulates during the irrigation season, and to pass the low flow of the river during the non-irrigation season. Conforming with the regulations of the Department of Fisheries of the State of Washington a fishway is provided in the north side of the river, having twenty bays and a rise of 2 ft. 1 in. between bays. The canal head-works are located on the extreme right of the dam and adjacent to the Northern Pacific Railway tracks. Two 12 by 11-ft. radial gates, motor-operated, control the flow of water into the Main Canal.

A full description of the Diversion Dam has been presented by Messrs. G. C. Imrie and A. A. Whitmore.[2]

THE CANALS

The canal system is composed of a single primary canal and two branch canals, as shown in Fig. 1(*b*).

[2] *New Reclamation Era*, March, 1931, p. 54.

Location.—From the Diversion Dam to the South Branch Head-Works, about a mile west of the Yakima River Crossing, the Main Canal of this project extends for a distance of about 25 miles. Its capacity at the point of diversion is 1 320 sec-ft. The variation in dimensions and hydraulic properties of the regular sections of the Main Canal is shown in Table 1. The side slopes in the regular sections of all canals were 1 on $1\frac{1}{2}$ in the unlined section and 1 on $1\frac{1}{4}$ in the lined sections. There were a number of special sections. All the reinforced concrete lining in the regular sections was 3 in. thick. About 50% of the Main Canal is concrete-lined although this lining is not all 3 in. thick and reinforced. Due to the topography of the ground, the canal follows along the side-hill slopes. In some places on account of very steep slopes it was impossible to build or to hold the lower banks. In these places combination sections of canal lining and bench flume were constructed. This section has a regular canal section on the inside and a vertical retaining wall on the outside. (See Fig. 5.) About 140 ft. of bench flume was also used. The walls of the bench flume and the outside wall of the combination sections were 11 ft. above the inside of the canal. These walls were 6 in. thick at the top and 14 in. at the bottom, reinforced on the inside and outside. The inside wall on the combination section was 6 in. of plain concrete and was poured against the trimmed rock face of the bluff. The hand-placed rock on the bluff side is shown in Fig. 5. There were also four canal sections termed "lined rock sections," which were excavated in shattered basalt formation, with side slopes of 1 on $\frac{1}{2}$. These sections were lined with an average thickness of 6-in. plain concrete with a minimum thickness of 2 in.

The earth sub-grade of the lined sections of canal is trimmed and tamped to accurate dimensions. The operation of trimming canal sections to prepare for lining (see Fig. 6) is paid for as a separate unit at so much per square yard.

The South Branch Canal has a capacity of 250 sec-ft. at the head-works and the flow is controlled at this point by electric, hoist-operated, radial gates. The Main Canal gate is 15 ft. long and 8 ft. high, and the South Branch gate is 8 ft. long and 8 ft. high. The South Branch Canal is about $14\frac{1}{2}$ miles long and its capacity at the end is 32 sec-ft. and, from that point, the total flow is taken in a main lateral which extends to the Yakima River Canyon (see Fig. 2). About $1\frac{1}{4}$ miles of the South Branch Canal is lined with 3-in. reinforced concrete slab; the remainder is unlined. The variation in dimensions and hydraulic properties of the regular canal sections is shown in Table 1.

The North Branch Canal is 36 miles long, $3\frac{1}{4}$ miles of which are ordinary, or combination, section. The variation in dimensions and hydraulic properties of the regular sections of the North Branch Canal is shown in Table 1. A part of the regular lined section is 4-in. reinforced concrete; the remainder of the regular lining is 3-in. reinforced concrete. There is a stretch of combination section between the South Branch Head-Works and the Yakima River Crossing. It has a vertical wall on the outside, 9 ft. high, 6 in. thick at the top, and 11 in. thick at the base, with reinforcement on the inside.

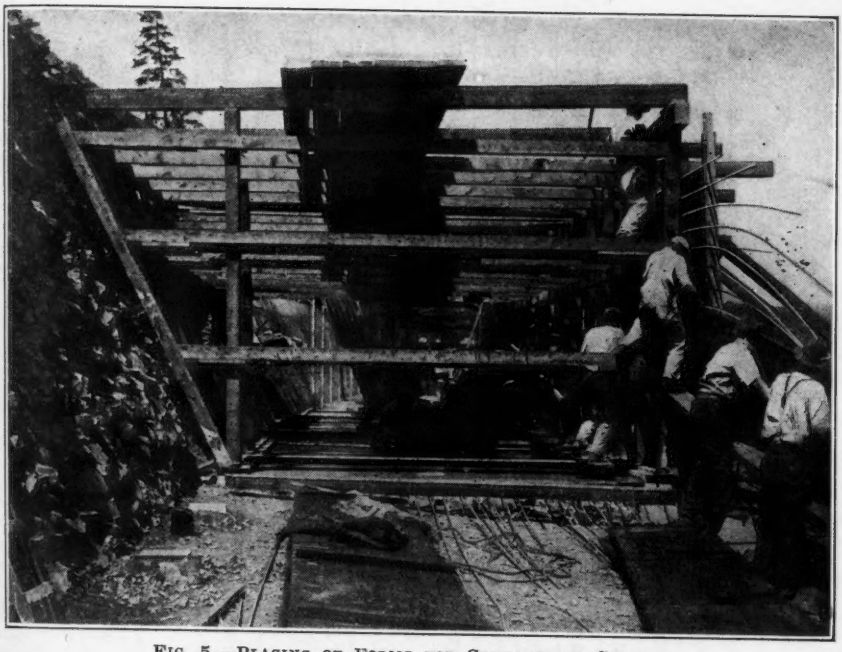

FIG. 5.—PLACING OF FORMS FOR COMBINATION SECTION

FIG. 6.—TRIMMING CANAL SECTION FOR LINING

TABLE 1.—Dimensions and Hydraulic Properties of Regular Sections Used in All Canals

Dimension	Main Canal		North Branch Canal		South Branch Canal	
	Unlined sections	Lined sections	Unlined sections	Lined sections	Unlined sections	Lined sections
Number of sections	5	9	19	6	15	4
Water depth, in feet	10.50 to 11.35	8.58 to 9.75	4.15 to 8.20	4.00 to 7.88	1.80 to 4.40	3.10 to 4.30
Lining depth, in feet	.....	9.93 to 11.00	.....	5.00 to 9.85	.....	3.85 to 5.30
Top of bank to water surface, in feet	5.00	4.00	1.0 to 3.0	2.0 to 4.0	1.2 to 2.5	1.5 to 3.0
Bottom width, in feet	30.00	11.0 to 12.0	12.0 to 30.0	6.0 to 10.0	6.0 to 16.0	4.0 to 6.0
Area, A, of stream, in square feet	480.38 to 534.00	189.66 to 236.00	81.84 to 346.86	44.00 to 148.47	15.66 to 76.86	24.41 to 48.91
Velocity, V, in feet per second	2.45 to 2.54	5.53 to 6.63	2.23 to 3.98	5.97 to 7.41	1.49 to 3.22	4.94 to 6.41
Flow, Q, in cubic feet per second	1 210 to 1 320	1 145 to 1 320	200 to 926	260 to 925	32 to 250	120 to 250
Assumed values of n in Kutter's formula	0.0225	0.014	0.0225 to 0.030	0.014	0.0225 to 0.030	0.014
Hydraulic radius, r, in feet	7.08 to 7.49	4.89 to 5.46	2.34 to 5.82	2.34 to 4.34	1.25 to 3.12	1.75 to 2.47
Slope, S, of hydraulic gradients	0.00009 to 0.0001	0.0003 to 0.00047	0.00015 to 0.001	0.0005 to 0.001	0.000315 to 0.001	0.0007 to 0.001

The inside wall of this combination section is poured with 4-in. reinforced concrete on a slope of 1 on $1\frac{1}{4}$. About $1\frac{1}{2}$ miles above the Johnson Creek Siphon there is a portion of unlined rock section. This section is in a deep cut and has a bottom width of 14 ft., with 4.75 ft. of water. The side slopes are 1 on $\frac{1}{2}$; a value of Kutter's n equal to 0.030 was used in the design of this section. The parts of the North Branch Canal that pass through porous gravel formations are covered with an earth blanket. To provide for the earth blanketing, the gravel is excavated from the bottom and sides to approximately 12 in. deeper than the limits of the normal canal section.

Design.—The Chezy-Kutter formula was used in design throughout. In the lined sections a value of $n = 0.014$ is used; and in the unlined sections $n = 0.0225$. In some unlined sections this was increased to 0.025 and 0.03, the higher values being used in some parts of the canals when the inside surface was very rough due to the presence of cement gravel and rock. Reference is made to the paper[3] entitled "The Hydraulic Design of Flume and Siphon Transitions," by Julian Hinds, M. Am. Soc. C. E., which gives plans and elevations of the transitions used by the Bureau of Reclamation.

Canal Lining Reinforcement.—The 3 and 4-in. reinforced concrete lining is provided with a berm at the top that extends horizontally with a 5-in. width. This creates an appearance of a 5-in. lining. The 3-in. lining is reinforced with $\frac{3}{8}$-in. round bars, spaced 24 in. apart transversely and 12 in. apart longitudinally. In the 4-in. lining, $\frac{1}{2}$-in. round bars are used with the same spacing. The $\frac{3}{8}$-in. bars are lapped 18 in. and the $\frac{1}{2}$-in. bars are lapped 24 in. In all other reinforced concrete work the length of splices is 40 diameters. Vertical construction joints extend across the base and up the slopes at about 14-ft. intervals. All longitudinal steel is continued through these joints without providing for expansion.

Canal Sub-Drainage.—All the lined sections of the canals are designed with a system of under-drains. The longitudinal drains are 4 to 10-in. vitrified clay or concrete sewer pipe laid in gravel-filled trenches in the center of the canal base. When side drains, or finger drains, were used the longitudinal drains were near the upper side and parallel to it. The longitudinal drain-tile was laid with open joints. In Fig. 7 it has been covered with tar paper. The trenches have a minimum depth of 10 in. and drain from 200 to 800 ft. from two directions into the lateral drains, which are 4 to 12-in. vitrified clay or concrete sewer pipe laid with cemented joints. The laterals drain the water from the longitudinal drains and extend beyond the canal prism and the waste banks. These lines have concrete cut-off and head-walls. (See Fig. 8.) The side drains or "finger drains" were spaced from 5 to 10 ft. These are trenches filled with gravel which extend from the longitudinal drains up the side slopes on the upper side of the canal. When completed they are covered with a thin layer of mortar to hold the gravel in place on the 1 on $1\frac{1}{4}$ slopes until the lining is poured.

Canal Side Banks.—A 14-ft. roadway on one side bank along the entire length of the Main Canal is intended for operation and maintenance purposes

[3] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1423.

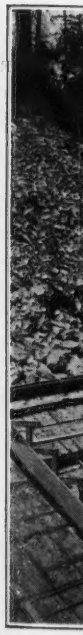

FIG. 7.—

FIG. 8.—

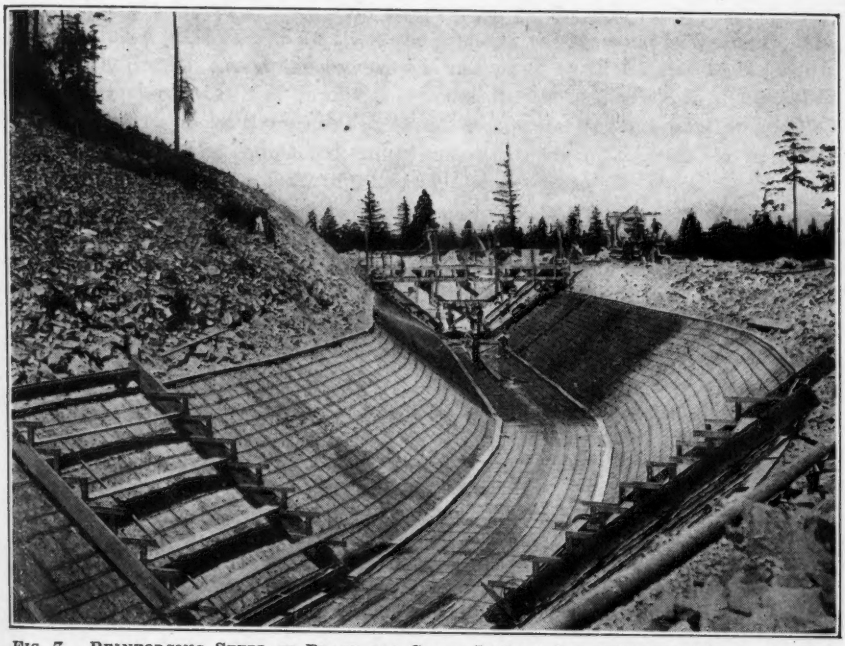

FIG. 7.—REINFORCING STEEL IN PLACE FOR CANAL LINING, SHOWING THE SCAFFOLDS USED IN PLACING STEEL

FIG. 8.—SIDE DRAINS, OR FINGER DRAINS, SHOWING LONGITUDINAL DRAINS NEAR BASE OF UPPER SLOPE

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only. On the North Branch this roadway is 14 ft. wide on the upper end, but is reduced to 12 ft. and 10 ft. as the capacity of the canal is reduced. On the South Branch Canal the roadway is narrowed to 12 ft. and 10 ft. which are the standard widths of bank tops. When the water surface in the canal is more than 3 ft. above the original ground, compacted embankment or puddled core is put in. The compacted embankment extends 1 ft. above the proposed water surface; it has a 6-ft. top, and side slopes generally equal to 1 on 1½. The compacted embankment is placed adjacent to the canal prism in successive horizontal layers, not thicker than 6 in. A roller weighing less than 2 000 lb. per lin. ft. does the compacting. These banks are usually built in advance of the regular side bank. Puddled core is placed vertically not less than 3 ft. in width, in the center of the canal bank. This core must extend 1 ft. above the proposed water level and about 3 ft. into undisturbed earth. All material is deposited and churned in water. In the lined portion of the canal the compacted embankment and puddled core are constructed to the top of the lining.

Inverted Siphons

The very rough topography made necessary the construction of a large number of siphons all of which may be classified in four types: (1) Monolithic reinforced concrete; (2) combination reinforced concrete and steel plate; (3) pre-cast lock-joint concrete pipe; and (4) concrete-lined tunnel siphons with both circular and horseshoe barrel. Taneum Siphon on the South Branch (Fig. 1(*b*) and Fig. 9) is one of the siphons constructed of lock-joint pipe; the siphons at Horseshoe and Morrison Canyons on the Main Canal (Fig. 1(*b*)) have steel pipe for the lower part where the hydrostatic head is relatively high; Dry Creek Siphon on the North Branch is one of the monolithic reinforced concrete structures.

The pressure tunnel siphon under the Yakima River passes through a stratum of basalt at an average depth of about 150 ft. below the ground. The tunnel lining is of three types, namely, plain concrete, reinforced concrete, and segmented steel plates with plain concrete lining.

Tucker Creek Siphon and the siphon at Station 275+50, South Branch, are square; the others are round. Table 2 gives the dimensions and hydraulic properties of all the siphons. In Column (3) the maximum head is the distance between the water surface in the inlet and the lowest point of the invert of the siphon. The horizontal distance in Column (4) is measured from the beginning of the inlet transition to the end of the outlet transition. Items Nos. 1 to 9 refer to the Main Canal; Items Nos. 10 to 16, to the South Branch Canal; and Items Nos. 17 to 22, to the North Branch Canal.

The reinforcing steel in the round cast-in-place siphons consists of from one to three layers, depending on the hydraulic head on the siphons. The transverse steel for one layer is bent in the form of an ellipse with the major axis horizontal, and is placed a minimum distance of 3 in. from the inside at the top and 3 in. from the outside on the horizontal axis. When more than one layer is used, the rings are circular. Each bar is a half ellipse with

TABLE 2.—DIMENSIONS AND HYDRAULIC PROPERTIES OF SIPHONS

		Dimensions				Hydraulic Properties					
Item No.	Location (see Figs. 1 and 2)	Maximum head, in feet	Horizontal length, in feet	Inside diameter, in feet	Thickness of shell, in inches	Area, A, of stream, in square feet	Velocity, V, in feet per second	Flow, Q, in cubic feet per second	Hydraulic radius, r, in feet	Slope, s, of hydraulic gradient	Assumed values of n in Kutter's formula
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	Tucker Creek	19.73	148	11.5 (square)	12	132.25	10.00	1 320	2.88	0.00215	0.014
2	Big Creek	30.55	188	12.5	13	122.71	10.76	1 320	3.13	0.00230	0.014
3	Little Creek	30.33	188	12.5	13	122.71	10.76	1 320	3.13	0.00277	0.014
4	Nelson Creek	59.67	488	12.0	13 and 12	113.10	11.66	1 320	3.00	0.00280	0.014
5	Peterson Creek	77.29	363	13.0	14 and 13	132.70	9.95	1 320	3.25	0.00183	0.014
6	Spexarth Creek	62.05	424	12.0	12 and 13	113.10	11.27	1 275	3.00	0.00262	0.014
7	Cle Elum Creek	125.33	588	12.0	15 to 12	113.10	11.07	1 252	3.00	0.00253	0.014
8	Horseshoe Canyon	141.53	862	12.1 12′–1″	18 to 12	114.67	10.29	1 180	3.02	0.00215	0.014
9	Morrison Canyon	239.99	1 050	12.1 12′–1″	15 to 12	114.67	10.00	1 145	3.02	0.00202	0.014
10	Station 91 +84	48.65	406	5.8 5′–10″	6 to 7	26.72	9.36	250	1.46	0.00453	0.014
11	Taneum Creek	207.10	3 326	4.5 (pipe) 5.0 (pipe)	4⅜ 5¼	15.90 19.63	9.12 7.39	145 145	1.125 1.25	0.00460 0.00265	0.012 0.012
12	Station 174 +10	76.11	464	4.5 (pipe)	4⅜	15.90	9.12	145	1.125	0.00460	0.012
13	Station 275 +50	13.68	163	5′–3″ 5.3 (square)	6 and 7	27.44	5.30	145	1.35	0.00164	0.014
14	Broadnax	54.76	508	4.5 (pipe)	4⅜	15.90	8.53	135	1.125	0.00400	0.012
15	Robinson Creek	14.08	191	5.0 (pipe)	5¼	19.63	5.35	105	1.25	0.00140	0.012
16	Manastash	19.45	486	3.0 (pipe)	3¼	7.07	7.78	55	0.75	0.01473	0.012
17	Hayward Canyon	158.15	545	11.2 11′–2″	11 to 16	97.93	9.47	925	2.79	0.00200	0.014
18	Sheep Dip Canyon	117.89	536	11.2 11′–2″	11 to 14	97.93	9.47	925	2.79	0.00200	0.014
19	Dry Creek	52.69	1 637	11.2 11′–2″	11 to 12	97.93	9.47	925	2.79	0.00200	0.014
20	Little Dry Creek	35.87	415	11.2 11′–2″	11	97.93	9.47	925	2.79	0.00200	0.014
21	Johnson Creek	115.86	1 815	6.0 (pipe)	6	28.26	9.92	280	1.50	0.00377	0.012
22	Siphon No. 6	43.46	356	6.0 (pipe)	6	28.26	9.92	280	1.50	0.00377	0.012

the 40-diameter laps staggered and placed on opposite diagonal quarter-points of the barrel. The 40-diameter laps of the longitudinal steel are staggered by spiraling the joints around the barrel.

Each of the siphon inlets is made up of two transitions, the first from the canal section to a square barrel entrance, and the second from the square entrance to a round barrel. (See Fig. 9.) The outlets reverse the order of the transitions. When the velocity is great enough to carry gravel in suspension, the entrances to siphons on lined canal sections are provided with gravel traps. These traps are 2 to 3-ft. depressions in the canal base, extending 40 to 80 ft. in an up-stream direction.

Construction joints in the form of 3 by 4-in. keyways, are spaced about 32 ft. apart. These joints are furnished with 4-in., 5-in., or 6-in. copper strips which are cast around the entire circumference of the barrel. The strip has a V-shaped crimp, $\frac{5}{16}$ in. deep, in the center, which allows for expansion and contraction of the barrel, without permitting leaks. When steel pipe is used in the lower part of the siphon, expansion is provided for by a sliding joint at each end of the pipe where it meets the concrete part.

The inside forms are supported on 1-in. square reinforcing bars bent in the form of a craddle and embedded at each end in a 1 ft. by 5 by 1.5-ft. concrete pier. As concrete is poured in the barrel section and around these supports they become a part of the structure, acting as anchors. The inside form supports are spaced on 8-ft. centers.

All monolithic siphons are furnished with an 8-in. blow-off valve at the lowest possible point for gravity drainage. The siphons with steel pipe are also furnished with a manhole near the bottom.

Wasteways

On all the canals provision is made for wasting the water during operation, or in case of emergency. In the Main Canal the water is controlled by standard radial gates, one in the canal and one in the wasteway chute. These gates are lifted by motor-driven hoists. The gates in the wasteways of the South Branch Canal are operated by hand. In the Main Canal there are wasteways at Peterson Creek Siphon, at Station 1146+30, and at Morrison Canyon Siphon. In the South Branch Canal, wasteways are located at Taneum Creek Siphon and Manastash Creek Siphon. Ten on the North Branch Canal are located at the following places: Yakima River Crossing, Naneum Creek, Johnson Creek, Cook Creek, Caribou Creek, Whipple Pumping Plant, one on each of the three laterals where they cross Badger Creek, and the wasteway to the Yakima River at the end of the lateral system.

Peterson Creek Wasteway is a 12 by 8-ft. flume, 280 ft. in length. The head-works are located in the inlet transition to the siphon. The flow of water is controlled by a 16-ft. radial gate in the wasteway and a 12-ft. gate in the canal.

The wasteway at Station 1146+30 consists of an 8 by 7-ft. concrete flume, 1 231 ft. long, with a drop in elevation of 340.5 ft. The slopes were 0.1800 for the upper part, and 0.6536 for the lower. The flume extends to the inlet

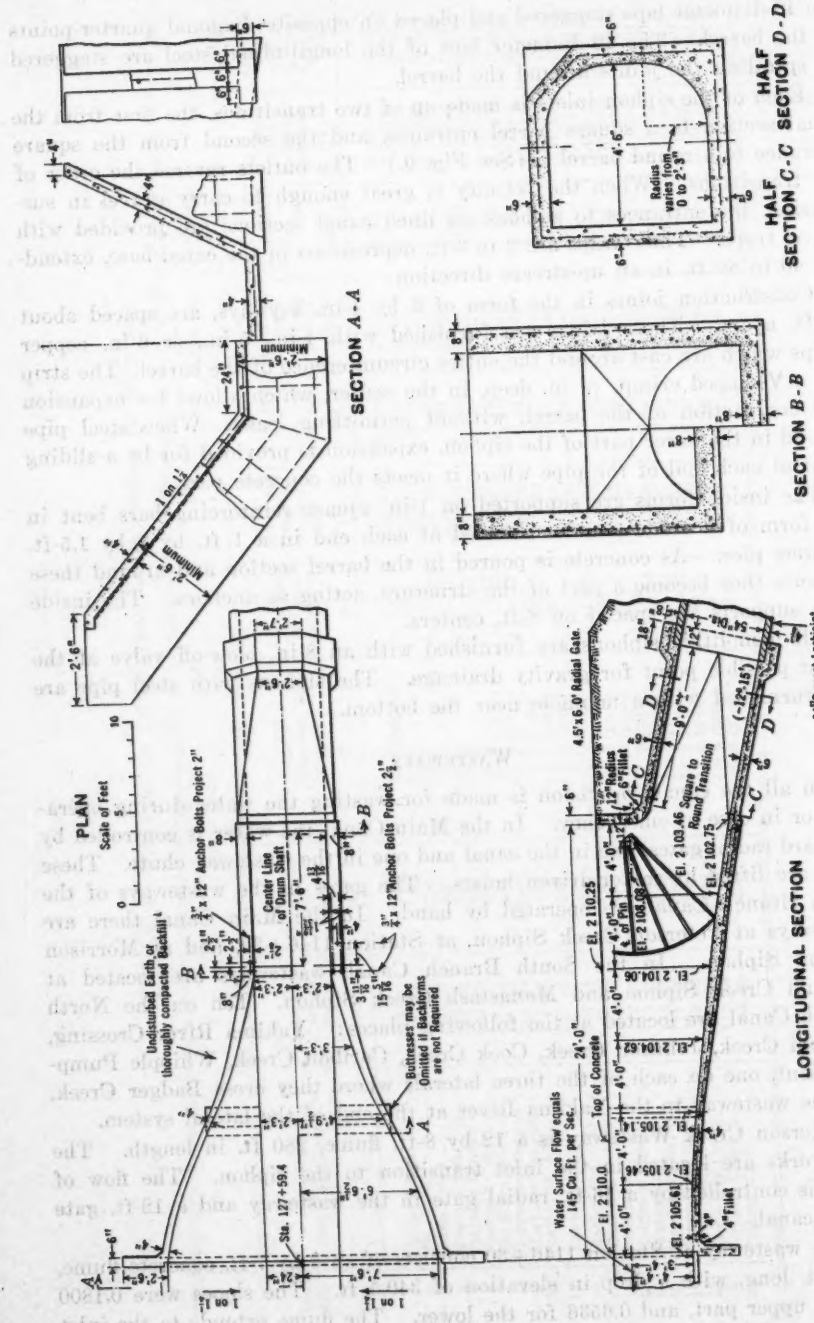


FIG. 9.—PLAN AND SECTIONS OF INLET TO TANEUM CREEK SIPHON

of a culvert which carries the water under the tracks of the Chicago, Milwaukee, St. Paul and Pacific Railway, and empties it into the Yakima River. Fig. 10 shows a plan of this structure. There are 20-ft. radial gates in the Main Canal and wasteway chute.

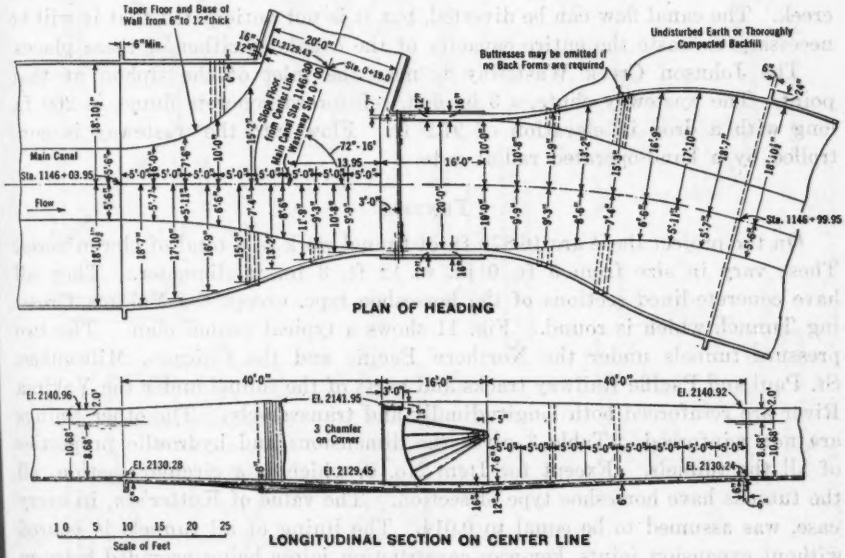


FIG. 10.—PLAN AND PROFILE OF WASTEWAY HEADING AT STATION 1146+30

Morrison Canyon Wasteway consists of two structures, the head-works connected to the inlet transition of Morrison Canyon Siphon, and the culvert under the railway tracks about ½ mile down the canyon (see Fig. 1(*b*)). A 100-ft. chute at the head-works ends at the top of a nearly vertical bluff about 125 ft. high. The water is allowed to drop over this bluff and then to follow the natural channel to the culvert. Flow is regulated with standard radial gates operated by electricity. A 20-ft. gate controls the wasteway and a 16-ft. gate controls the Main Canal.

Taneum Creek wasteway on the South Branch, is 2 821 ft. long, of which 1 326 ft. is lined with concrete and has a drop in elevation of 202.8 ft. The chute is a lined canal section with a bottom width of 4 ft, a depth of 3 ft. 3 in., and with 1 on 1½ slopes. This chute empties into a stilling-pool, and the water is carried off in an unlined ditch. The wasteway and canal are furnished with hand-operated gates. At Station 14+00 on the wasteway, the Bruton (private) Ditch crosses under the wasteway channel in a 24-in. pre-cast pipe siphon (see Fig. 2(*b*)).

The Manastash Wasteway on the South Branch Canal is 120 ft. long. It has a relatively gradual fall and is constructed of 36-in. pre-cast concrete pipe. The inlet is in the head-works of the Manastash Siphon, and the flow is regulated by hand-operated gates. As this is the end of the South

FIG. 9.—PLAN AND SECTIONS OF INLET TO TANEUM CREEK SIPHON
SECTION B-B
LONGITUDINAL SECTION
54" Precast Lockjoint Concrete Pipe

Branch Canal, there is an 8-ft. Cipolletti metal-crest weir installed in a baffle wall in the siphon transition. From this point the flow of the South Branch Canal is carried in a main lateral (see Fig. 2).

In the case of Naneum Creek, Cook Creek, and Caribou Creek Wasteways, the base of the canal is approximately at the same level as the bed of the creek. The canal flow can be diverted, but it is not anticipated that it will be necessary to waste the entire capacity of the canals at either of these places.

The Johnson Creek Wasteway is near the inlet of the siphon at that point. The wasteway chute, a 3 by 6-ft. reinforced concrete flume, is 260 ft. long with a drop in elevation of 97.2 ft. Flow into the wasteway is controlled by a hand-operated radial gate.

Tunnels

On the project there are 16 878 ft. of tunnel work in a total of eleven bores. These vary in size from 5 ft. 0 in. to 12 ft. 3 in. in diameter. They all have concrete-lined sections of the horseshoe type, except the Yakima Crossing Tunnel, which is round. Fig. 11 shows a typical tunnel plan. The two pressure tunnels under the Northern Pacific and the Chicago, Milwaukee, St. Paul and Pacific Railway tracks and parts of the tunnel under the Yakima River are reinforced both longitudinally and transversely. The other linings are not reinforced. Table 3 gives the dimensions and hydraulic properties of all the tunnels. Except for Item No. 6, which is a circular section, all the tunnels have horseshoe type of section. The value of Kutter's n, in every case, was assumed to be equal to 0.014. The lining of all tunnels is poured without expansion joints, key-way construction joints being provided between pours.

TABLE 3.—Dimensions and Hydraulic Properties of Tunnel Sections

Item No.	Name	Dimensions					Hydraulic Properties				
		Length, in feet	Width and height, in feet	Lining Thickness, in Inches		Water depth, in feet	Area, A, of stream, in square feet	Velocity, V, in feet per second	Flow Q, in cubic feet per second	Hydraulic radius, r, in feet	Slope, s, of hydraulic gradient
				Minimum	Average						
	Main Canal										
1	Tunnel under Chicago, Milwaukee, St. Paul, and Pacific Railway	180	12′-3″ / 12.2	8	12	Full	124.40	10.61	1 320	3.11	0.00222
2	Tunnel under Northern Pacific Railway	304	12′-3″ / 12.2	8	12	Full	124.40	10.61	1 320	3.11	0.00222
	South Branch										
3	Tunnel No. 1	2 000	6.25	6	10	5.25	29.20	8.59	250	1.91	0.00270
4	Tunnel No. 2	1 390	5.0	5	8	3.90	17.20	7.86	135	1.53	0.00300
	North Branch										
5	Rocky Point	965	11.9	6	12	9.86	104.75	8.84	925	3.64	0.00125
6	Yakima River Crossing	3 125	9′-3″ / 8.3	9	15 and 18	Full	87.20	13.77	925	2.31	0.00538
7	Tunnel No. 1	1 686	11.7	6	12	9.66	101.00	9.16	925	3.59	0.00137
8	" " 2	1 025	11.7	6	12	9.66	101.00	9.16	925	3.59	0.00137
9	" " 3	2 276	11.7	6	12	9.66	101.00	9.16	925	3.59	0.00137
10	" " 4	482	11.2	6	12	9.1	90.99	8.51	775	3.42	0.00123
11	" " 5	3 445	6.7	6	10	5.53	33.08	8.47	280	2.05	0.00240

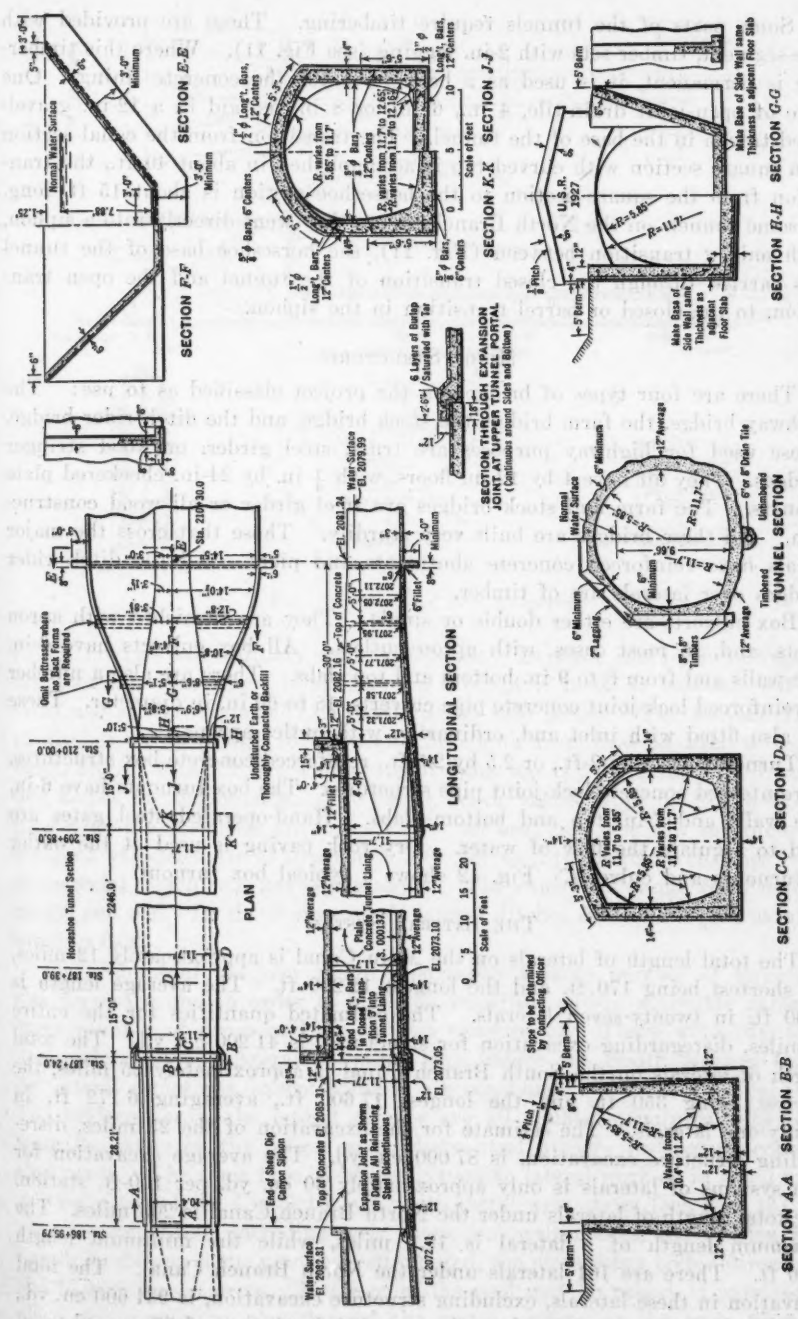


FIG. 11.—PLAN AND TYPICAL SECTIONS, TUNNEL NO. 3, NORTH BRANCH CANAL

Some parts of the tunnels require timbering. These are provided with five-segment, timber sets with 2-in. lagging (see Fig. 11). Where this timbering is permanent, it is used as a back form for the concrete lining. One line of open-joint drain-tile, 4 in., 6 in., or 8 in., is laid in a 12-in. gravel-filled trench in the base of the tunnel. The transition from the canal section to a square section with curved top is accomplished in about 40 ft., the transition from the square section to the horseshoe section is about 15 ft. long. In some tunnels on the North Branch, where they went directly into a siphon, with only a transition between (Fig. 11), the horseshoe base of the tunnel was carried through the closed transition of the tunnel and the open transition, to the closed or barrel transition in the siphon.

Minor Structures

There are four types of bridges on the project classified as to use: The highway bridge, the farm bridge, the stock bridge, and the ditch rider bridge. Those used for highway purposes are truss, steel girder, or wood stringer bridges. They all have 4 by 12-in. floors, with $\frac{1}{4}$ in. by 24-in. checkered plate runners. The farm and stock bridges are steel girder or all wood construction. All these bridges are built very sturdily. Those that cross the major canals have reinforced concrete abutments and piers. All the ditch rider bridges over laterals are of timber.

Box culverts are either double or single. They are furnished with apron inlets, and, in most cases, with apron outlets. All box culverts have 6-in. side walls and from 6 to 9-in. bottom and top slabs. There are also a number of reinforced lock-joint concrete pipe culverts, 18 to 60 in. in diameter. These are also fitted with inlet and, ordinarily, with outlet aprons.

Turnouts are 2 by 2-ft., or 2.5 by 2.5-ft., reinforced concrete box structures, or reinforced concrete lock-joint pipe structures. The box turnouts have 6-in. side walls and 7-in. top and bottom slabs. Hand-operated steel gates are used to regulate the flow of water. Dry rock paving is used at the outlet of turnouts and culverts. Fig. 12 shows a typical box turnout.

The Lateral System

The total length of laterals on the Main Canal is approximately 12 miles, the shortest being 170 ft. and the longest 12 200 ft. The average length is 2 400 ft. in twenty-seven laterals. The estimated quantities for the entire 12 miles, disregarding excavation for structures, is 41 200 cu. yd. The total length of laterals on the South Branch Canal is approximately 25 miles, the shortest being 350 ft. and the longest 17 600 ft., averaging 6 172 ft. in twenty-one laterals. The estimate for the excavation of the 25 miles, disregarding structure excavation, is 87 000 cu. yd. The average excavation for both systems of laterals is only approximately 60 cu. yd. per 100-ft. station. The total length of laterals under the North Branch Canal is 205 miles. The maximum length of a lateral is 19.1 miles, while the minimum length is 90 ft. There are 161 laterals under the North Branch Canal. The total excavation in these laterals, excluding structure excavation, is 931 600 cu. yd., or an approximate average excavation per 100-ft. station of 86 cu. yd.

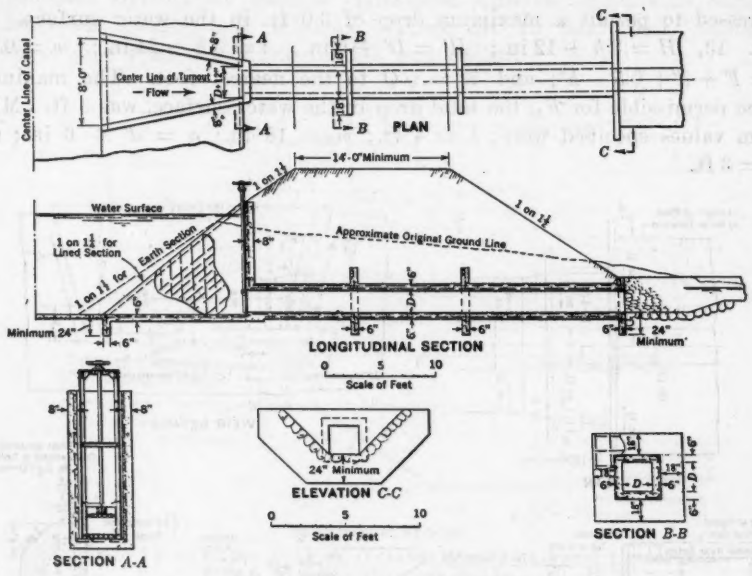


FIG. 12.—TYPICAL BOX TURNOUT

All laterals are ordinary unlined ditches. In most cases the side slopes are 1 on $1\frac{1}{2}$, with two side banks. An attempt is made to balance "cut" so as to use all the material excavated in the side banks. The bottom width of ditches varies from 3 to 12 ft. and the depth from base to ditch to top of bank, from 1.5 to 5.5 ft. The width of bank top is 3 or 4 ft., with 1 on $1\frac{1}{2}$ side slopes on the outside. The water depth varies from 0.4 ft. to 3.0 ft. As a rule, the velocities are about 2.0 ft. per sec., but in a few instances the designed velocities are as great as 3.7 ft. per sec., the higher velocities being in material which will not easily scour. The capacities range from 6 to 75 cu. ft. per sec. In the design of all lateral ditches Kutter's n was assumed equal to 0.025.

On account of the very rough topography a large number of structures was necessary and in order to hold the velocities within allowable values, a large number of drops and chutes was constructed. There were fifty-three drops alone in one lateral $2\frac{1}{2}$ miles long, under the South Branch Canal. All lateral structures, except bridges, are constructed of reinforced concrete. The bridges across the lateral ditches are all timber structures with wood mudsills; some of the longer spans have steel girders resting on concrete abutments. The same type of bridge is used in lateral work as in canal work. All weirs are of the Cipolletti type, with metal weir plates (see Fig. 13). In all weir structures, including weir drops and weir chutes, the wooden face-board, as shown on the plan, was eliminated, the plate being bolted directly on the face of the concrete. The floor of the pool may be raised, if required, to place the maximum elevation of the water surface at the weir crest, or

depressed to permit a maximum drop of 3.0 ft. in the water surface. In Fig. 13, $H = 3\,h + 12$ in.; $H' = D' + 6$ in.; $1 = 3\,h + 24$ in.; $m = 0.3\,l$; $n = F + d' + 0.5 - h'$; and $W = \sqrt{Q}$ to the nearest foot. The maximum value permissible for F_x, the total drop in the water surface, was 3 ft. Minimum values specified were: $l = 4$ ft.; $m = 18$ in.; $n = d' + 6$ in.; and $W = 3$ ft.

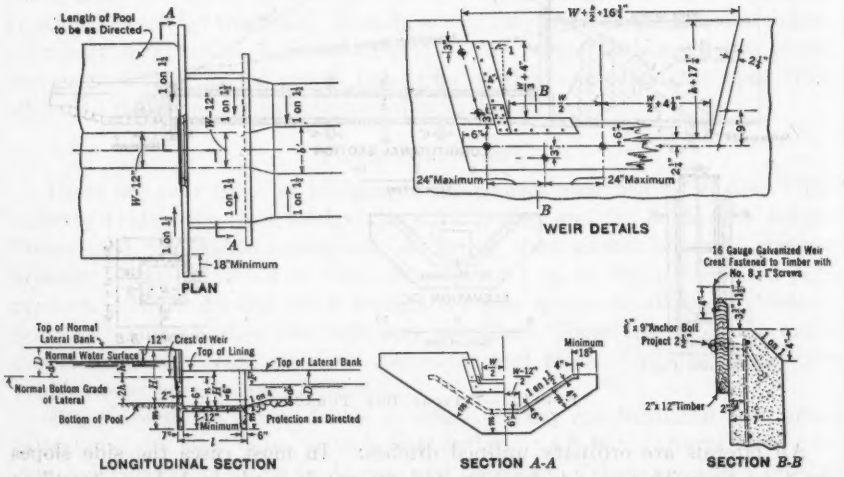


FIG. 13.—PLAN OF STANDARD WEIR, CAPACITY 70 CUBIC FEET PER SECOND

The checks are similar in construction to the weirs, except that the first wall is fitted with grooves for the insertion of the flash-boards. In the weirs this wall has the trapezoidal notch and weir plate. Three types of drops are used: The inclined, the inclined weir, and the inclined drop with flash-board control. The last two are fitted at the upper end as weirs and checks, thus eliminating these separate structures. There are only very minor differences in design of the three structures, except the addition of the wall that carries the weir plate or is notched for the flash-boards. Fig. 14 is a typical plan of the inclined drop. Note that $B' = B + 1\frac{1}{2}d$; $e = \frac{3}{4}\,(1\frac{1}{4}\,H' - 1\frac{1}{2}\,H)$; $H = 1.2\,d + 6$ in.; $H' = P + 1.2\,d' + 0.5\,F + 12$ in.; $l = 1.5\,(F + P)$; $l' = (F + 1.5d')$; and $P = \frac{1}{4}\,(\sqrt{d'\,F} + d')$. The following values are the maximum permissible; $B = b$; $B' = b$; and $F = 15$ ft. The minimum permissible values are $H = 18$ in. and $l' = 4$ ft.

Chutes are either paved with dry rock or are lined with reinforced concrete. The flow through sub-lateral turnouts is controlled by hand-operated gates and the barrels of these turnouts are reinforced "lock-joint" concrete pipe, 18 to 48 in. in diameter. The intakes and pipe collars are monolithic except on one South Branch lateral where, for experiment, they were pre-cast slabs, with mesh reinforcement at the joints arranged so that they joined together when placed, in the same manner as "lock-joint" pipe. The slabs are 3 in. thick and are reinforced with mesh and $\frac{3}{8}$-in. round bars. A number

of low-head siphons are used in connection with the laterals as crossings under roads, canals, and railroads. These are all reinforced "lock-joint" concrete pipe with monolithic intakes and outlets.

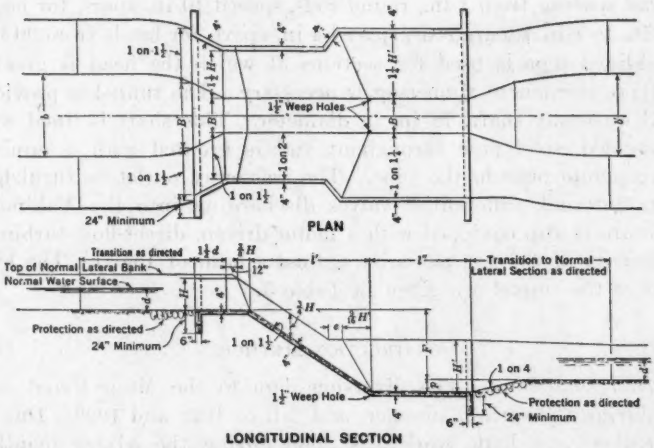


FIG. 14.—TYPICAL PLAN OF INCLINED DROP; MAXIMUM CAPACITY, 70 CUBIC FEET PER SECOND

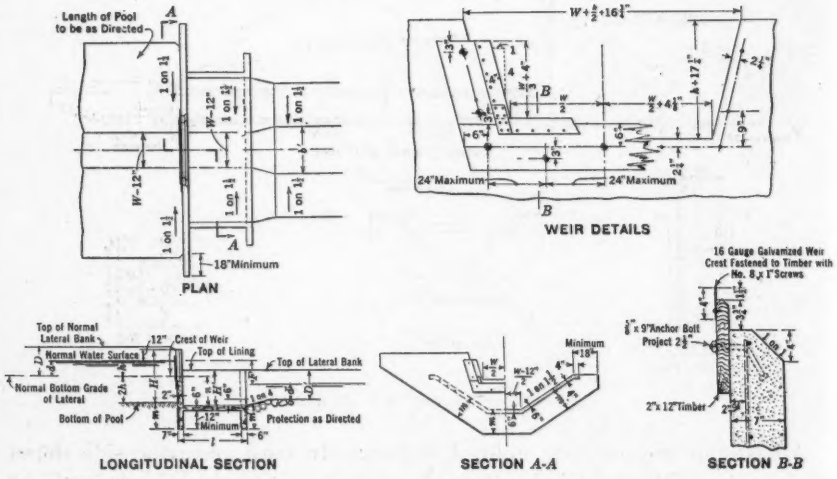
THE YAKIMA RIVER CROSSING

About 26 miles from the diversion dam, the Main Canal crosses the tracks of the Chicago, Milwaukee, St. Paul, and Pacific, and the Northern Pacific Railways, the proposed location of the Swauk Creek Logging Railway, the Yakima River, Swauk Creek, and the proposed location of the Inland Empire Highway (see Fig. 1). The difference in elevation of the water surface in the canal above the location and the normal water surface in the river is 390.8 ft. The railway tracks are near, and a few feet above, the water surface in the river. Between the 2 100-ft. contours (which is approximately the elevation of the canal) on either side of the river, the horizontal distance is about 3 125 ft. Fig. 15 shows the location of the tunnel. The vertical waste-way shaft in the photograph was also used as a construction shaft.

Thirteen test holes were drilled at the location, along the line of the proposed tunnel. These tests indicate that, in general, the bore will be through shattered basalt formation. The tunnel section is round, 9 ft. 3 in. in diameter, and lined with concrete throughout. Its horizontal length is 3 125 ft. The first 350 ft. has a slope of —1.428, the lower part has a slope of — 0.003 and + 0.003, and the outlet (334 ft.) has a slope of + 1.428. Of course, these are not the hydraulic slopes. The maximum head is 533.2 ft. measured from the water surface at the intake to the lowest point of the tunnel.

Three types of lining sections are to be used varying with the material encountered. The section lined with plain concrete, to be used in stable rock, has an average thickness of 15 in. with a minimum of 9 in. In the steel-lined section the pipe segments are pressure grouted on the outside, with an 18-in. reinforced concrete lining on the inside. The reinforced concrete section

depressed to permit a maximum drop of 3.0 ft. in the water surface. In Fig. 13, $H = 3\,h + 12$ in.; $H' = D' + 6$ in.; $1 = 3\,h + 24$ in.; $m = 0.3\,l$; $n = F + d' + 0.5 - h'$; and $W = \sqrt{Q}$ to the nearest foot. The maximum value permissible for F_x, the total drop in the water surface, was 3 ft. Minimum values specified were: $l = 4$ ft.; $m = 18$ in.; $n = d' + 6$ in.; and $W = 3$ ft.



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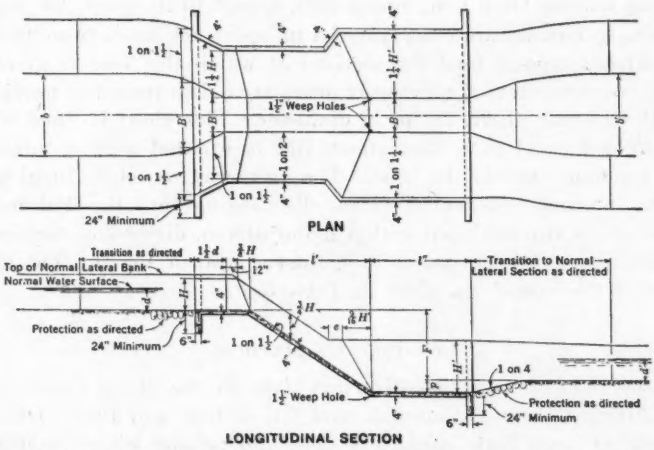

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has an average thickness of 15 in. and a minimum, of 9 in. (see Table 3, Item No. 6). For the longitudinal reinforcement, forty-eight $\frac{5}{8}$-in. round bars were specified. The transverse reinforcements consist of hoops varying in size and spacing from 1-in., round rods, spaced 10 in. apart, for heads from 30 to 40 ft., to 1-in. square rods, spaced 4 in. apart, for heads from 90 to 100 ft. The steel-lined pipe is used for sections at which the head is greater than 100 ft. where permanent timbering is necessary. The tunnel is provided with a vertical wasteway shaft, 78 in. in diameter. This shaft is lined with $\frac{3}{4}$-in. hammer-welded, steel pipe throughout and is grouted with a minimum of 9 in. of concrete outside the pipe. The wasteway outlet is furnished with two 42-in. internal, differential valves discharging into the Yakima River. The structure is also equipped with a motor-driven, direct-flow turbine pump, with a capacity of 600 gal. per min. against a head of 155 ft. The hydraulic properties of the tunnel are given in Table 3.

Construction Methods

The Diversion Dam.—The diversion dam to the Main Canal was constructed during the spring, summer, and fall of 1928 and 1929. Due to very severe weather very little work was done during the winter months. No serious high water occurred during the construction period. A timber flume, 12 ft. wide and 9 ft. deep, was used to carry the normal flow of the river during the early period of construction. The inlet floor of this flume was at Elevation 2 142.5. The upper and lower coffer-dams were built of rock and gravel, to Elevations 2 155 and 2 143, respectively. They were placed directly on the sand and gravel bed of the river. This resulted in a larger flow through the coffer-dams than could be cared for by pumps, and necessitated the driving of interlocking sheet-steel piling and the addition of blanketing material. When the structure of the dam was above the high-water stage the temporary timber flume was removed, and the river passed through an opening left in the base of the dam near the north side. This was closed when the sluice-gates had been placed. The base of the dam was set in 3 to 4 ft. of solid rock, which was encountered under a stratum of 6 to 15 ft. of sand, gravel, and boulders. The cut-off wall was carried 5 ft. below the base of the dam. In the center of the key-way, grout holes were spaced in a single line, 5 ft. apart. Additional holes were drilled in soft or seamy spots in the rock. The depth of the grout holes ranged from 25 ft. in the main part of the dam to 10 ft. under the abutments. In addition, 20 grout holes were drilled in the south half of the spillway section below the key-way. There were 107 holes drilled with a total length of 2 233 ft. They were $2\frac{1}{2}$ in. in diameter at the top and tapered to 1 in. in diameter at the bottom. As much as $38\frac{1}{2}$ cu. ft. of grout was used in one hole, while some holes would take no grout. The average was 4.75 cu. ft. per hole. The grouting was done after a minimum depth of 5 ft. of concrete had been poured, and forced in under a pressure of 100 lb. per sq. in.

The concrete mix for the interior of the dam was 1:2.75:6.25, with a slump of 2 to 3 in. and a water cement ratio of 1.10. For the exterior 3 ft.

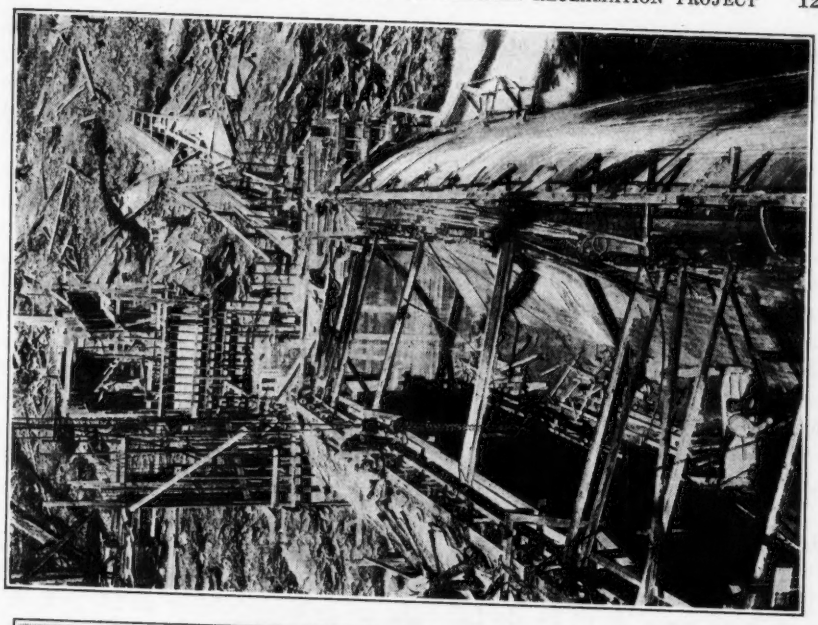

FIG. 16.—CONSTRUCTION VIEW OF DIVERSION DAM

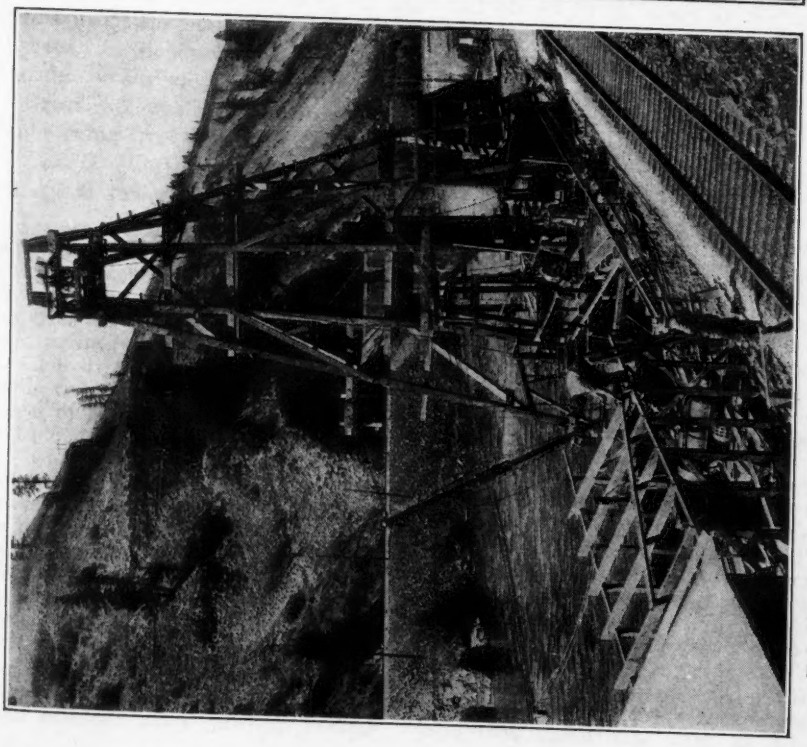

FIG. 15.—LOCATION OF THE YAKIMA RIVER PRESSURE TUNNEL

of the dam, a mix of 1.5 : 2.75 : 6.25, with a slump of 3 to 5 in. and a water-cement-ratio of 0.80, was used. Of diatomaceous earth, 3 lb. per sack of cement was used for both mixes. The maximum size of aggregate was 3 in. These mixes were quickly interchangeable at the mixer. By the addition of an extra sack of cement to each mixer batch, of the mixer used, the interior mix could be changed to the exterior mix. In the fish ladder, head-works, and other parts, where greater strength was required, the mix was 1 : 2.4 : 3.6, with a 1¼-in. maximum size of aggregate. An admixture of 3% was used with a slump of 4 to 6 in. and a water-cement-ratio of 0.93. In binding successive horizontal lifts about ¾ in. of the old concrete was removed with picks, and the surface was thoroughly cleaned with air, steel brushes, and water. Then a 1-in. layer of 1.5 : 2.75 grout was added and brushed with steel brushes. This was followed with a 6-in. layer of 1.5 : 2.75 : 6.25 concrete. The concreting was then continued with the 1 : 2.75 : 6.25 mix. An efficient arrangement for pouring concrete was made. Gravel, sand, and cement were chuted to the mixer, and the mixed concrete was chuted to the larger part of the work. The total concrete in the structure was 5 706 cu. yd.

Metal-lined forms, 5 by 6 ft., were used. These were anchored with ½-in. anchor rods. The pours varied in thickness from 4 to 10 ft. and horizontal key-ways were provided between them. These key-ways were formed with two 2 by 12-in. planks standing on edge and placed 18 in. apart at the top and 24 in. at the bottom. They were held in place by a steel rail bent in a U-shape. A richer mix with smaller aggregate than the 3 in. was used in pouring the key-ways.

Great care was exercised in placing the drum-gate in order that this gate would swing freely. Fig. 16 is a construction view of the dam.

Canals.—Earthwork was divided into three classes for payment, both in canal and structure excavation. The specifications are as follows:

"*Class No. 1.*—Material that can be plowed to a depth of six inches or more with a 6-horse or 6-mule team, each animal weighing not less than 1 400 pounds, attached to a suitable plow, all well handled by four men; also material that is loose and can be handled with team-drawn scrapers, and material that can be excavated efficiently, without blasting, by power shovels or dragline excavators of recognized standard manufacture, equipped with suitable dippers or buckets having minimum capacities of ¾ and 1 cubic yard, respectively, well handled by skilled operators; also all boulders or pieces of rock not exceeding 2 cubic feet in volume, that are detached or are embedded in Class 1 material.

"*Class No. 2.*—All material other than Class 1 and Class 3, including, but not restricted to, indurated material of all kinds, such as hard pan, cemented gravel and soft or disintegrated rock, which can not be excavated efficiently by power shovels or dragline excavators of the size and class above described, or by the use of plows and team-drawn scrapers, until loosened by blasting; also, all boulders or detached pieces of rock more than 2 cubic feet and not exceeding one-half cubic yard in volume, and all slide rock or similar loose rock occurring in masses which can not be handled efficiently with team-drawn scrapers, except pieces exceeding one-half cubic yard in volume.

"*Class No. 3.*—All solid rock in place which can not be removed until loosened by blasting, and all boulders or detached pieces of solid rock more than one-half cubic yard in volume. Solid rock under this class, as dis-

tinguished from soft or disintegrated rock under class 2, is defined as sound rock of such hardness and texture that it can not be effectively loosened or broken down by hand drifting picks."

Canal sections were excavated with draglines, shovels being used only in rare instances. The bucket capacity varied from 1 cu. yd. to 2½ cu. yd. Some cuts as deep as 65 ft. were excavated with two draglines, one of them passing dirt to the other higher up.

Concrete for the lining of the Main Canal and the North Branch Canal was mixed in a 3-bag, paving mixer. The slopes were paved by the use of a machine made expressly for that purpose. It had a structural steel framework adjustable to the size and form of the canal section and was equipped with a movable steel form on each side the length of one panel and extending up the slope about 3 ft. The concrete was poured between the form and the trimmed canal slope, the form being held in place so that a lining of the proper thickness was secured. The steel form could be pulled up the slope by a winch mounted upon it, as the concrete was poured and tamped. The bottom was poured last, well tamped, and finished. When the machine had been moved in place for the next panel, the preceding one was finished by hand.

Smaller mixers were used on the South Branch Canals and on lateral chutes and for this work the proper thickness was obtained with screed boards. Fig. 17 shows alternate panels being poured on the South Branch lining. This method was not found to be successful because of the expansion of steel in the open panels. On this work concrete was mixed at a central plant and hauled in trucks. One contractor used this system with some success on lateral structures, but most of them were poured on location, with small ½-bag or 1-bag mixers. Fig. 18 shows a lateral chute being constructed.

The reinforcing steel was cut and bent on the job and was placed and wired ahead of the concreting. It was supported in place by small concrete blocks made to the proper thickness.

In the Combination Section a novel arrangement of panel forms was used (see Fig. 5). The timbers were bolted so that a very rigid structure was obtained, and, at the same time, they could be assembled and taken apart quickly. The lagging was made in large panels bolted together and lined with metal. Arrangements were made so that concrete was poured every third day and the forms were moved up and made ready for the next pour during the intervening days.

Ordinarily, in pouring canal lining on the Main Canal and on the North Branch Canal, the gravel, sand, cement, and admixture were proportioned at the bunkers and hauled dry to the job in batch trucks, three to six 3-bag batches per truck. The proportioning as well as the placing of the concrete in the canal lining was under the direction of an inspector. In most cases both sand and gravel were weighed, but in some instances the gravel was measured by volume.

A full description of the construction of the main canal lining has been presented by Messrs. Arthur Ruettgers and A. A. Whitmore.[4]

[4] *Journal*, Am. Concrete Inst., October, 1930, p. 117.

FIG. 17.—POURING CONCRETE LINING ON SOUTH BRANCH CANAL. ALTERNATE PANELS BEING POURED AND SCREEDED TO PROPER THICKNESS.

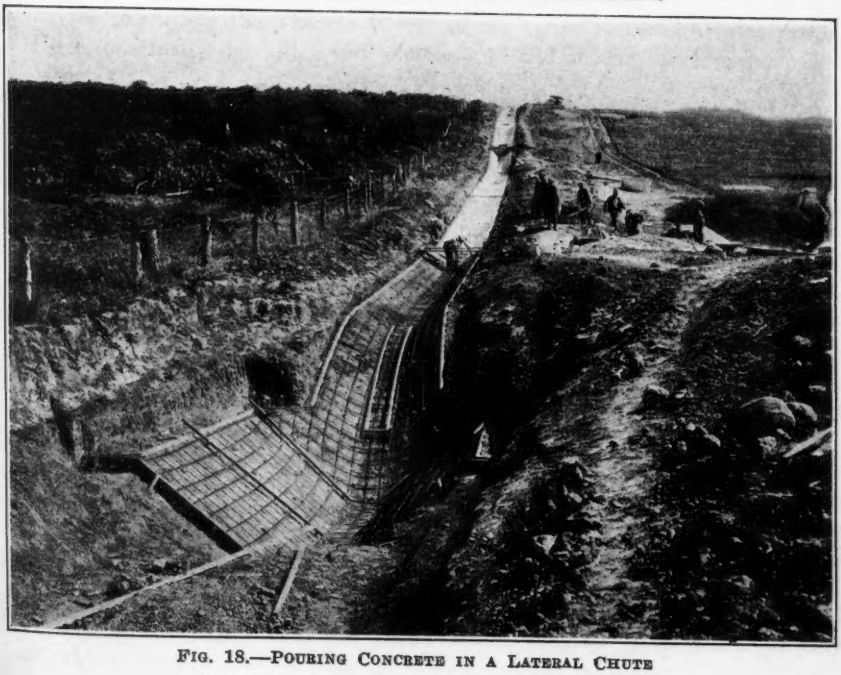

FIG. 18.—POURING CONCRETE IN A LATERAL CHUTE

Tunnels.—With a few exceptions the formation through which tunnels were driven was glacial, cemented, clay gravel. Very little permanent timbering was necessary in the construction work; no tunnel had more than one construction shaft; no difficulties were encountered in drilling and blasting and no serious slides and soft spots were encountered. The trimming was left very rough because all the tunnels were to be lined with concrete. When loose rocks dropped, or when the excavation was wide, the excessively large pockets thus left were filled with sound spalls held by timbering or tamped in place with concrete. Muck was loaded into small dump cars by hand and the cars hauled away by horse or mule.

In the Rocky Point Tunnel, an endless electrically operated dragline was used to load the muck train. The cable ran through one pulley connected to an eye-bolt driven in the heading; the other end was wound around a reversible electrically driven drum. The bucket was pulled up an incline to a tramway over the muck train, where it was automatically dumped. The action of the drum was then reversed, the empty bucket being pulled down the muck pile and loaded on the return. The incline and tramway could be moved up as the work progressed. The muck train was hauled by horses or mules on 36-in. gauge track. All permanent timbering was composed of 5-segment timber sets, the size in the smaller tunnels being 6 by 6 in., and in the larger tunnels, 8 by 8 in., with 2-in. lagging. Small electrically driven blowers were used for ventilating purposes and the air was good in all tunnels. Except for the two tunnels under the railway tracks near the diversion dam and the one under the Yakima River, all the tunnels are high in elevation. Hence, practically no water was encountered and the drainage problem was simple.

In most of the larger tunnels the floor was poured by hand and provided a support for the collapsible steel forms. A key-way joint was placed between the floor and the side walls. The forms had timber lagging and the concrete was shot in place behind them with air, the pipe for this purpose being placed inside at the top of the forms and then pushed to the back or to the beginning of the day's pour. Jack hammers were used on the inside face of the forms and no other kind of tamping was done. Open inspection holes were left in the lagging and closed as the concrete pouring progressed. The concrete was mixed outside the tunnel near the most accessible end for delivery of material and was transported to the "gun" by means of a gas locomotive. Only two crews of men were employed, one to set the forms during the night and the other to pour the concrete during the day shift.

The concrete in the smaller tunnels was placed by hand. The same type of forms was used. The side walls were poured first and then the floor, which provided additional head-room. No reinforcement steel was used in the concrete lining of the "no-pressure" tunnels, except in the portal transitions.

Siphons and Wasteways.—The excavation in the majority of siphons and wasteways was hand work, the classification ranging from basalt to clay gravel.

All form work in these structures was timber, but the siphon forms were bolted together in order that they could be re-used. The usual procedure was to pour the siphon barrel on one side of the canyon and then move the forms to the other side and pour there. The inside forms, without bracing, can be seen in Fig. 19. On most siphons the concrete was mixed at the top and chuted in to place, but on some structures, material could be more easily assembled at the bottom. An aerial cable line was used at two siphons. No elevating towers were used on any structure. The larger siphons contained approximately 1 300 cu. yd., with the exception of Dry Creek Siphon which contained 2 640 cu. yd. The main difficulty in pouring the barrels was in getting the expansion joints properly installed as the men pouring the concrete did not realize the extreme importance of these joints.

The placing of steel pipe at the bottom of Morrison Canyon Siphon presented a difficult problem. Particularly, it involved the question of just how much machinery it was economical to assemble for a small job. Pipe was placed by means of gin-pole rigging, but the canyon sides are so steep and treacherous, being covered with loose, sliding rock, that this was a very dangerous and slow method. (See Fig. 20.) It was nearly impossible for the contractor to get good steel men who would stay on the job, due partly to the nature of the work and partly to the location.

All siphon forms were of timber. The inside forms were 8-segment, circular forms, with ribs spaced about 24 in.; the ribs were alternately braced with two 4 by 4-in. posts at right angles which, in turn, rested on 4 by 4-in. walers. Two walers were in the base of the forms and were bolted to the inside form supports to hold the inside forms in place. The lagging was 2 by 6-in., tongue and groove, bevel-edged material. The inside forms for the siphons were usually built complete for one side of the canyon, and three or more, panels of the outside forms were used. A panel of outside forms was moved ahead each third day, pouring being done at 3-day intervals. The outside forms were constructed so that they could be adjusted for changes in shell thickness. Inspection holes were cut in the inside lagging where it was necessary to see that the concrete was thoroughly tamped.

In pouring the lower part of the wasteway at Station 1146+30 and the culvert under the railroad, it was necessary to construct a track on top of the structure. The mixed concrete was carried in small cars controlled by an endless cable. The slope of the wasteway on the lower end is 0.65. The mixed concrete in the remainder of the structure was poured by means of chutes and in wheel-barrows. The floor was poured first, and then the side walls, forms being built on top of the concrete floor after it had set.

Method of Controlling Quality of Concrete

About 150 000 cu. yd. of concrete were produced at an expenditure of about $4 500 000 on this project. Consequently, the problem of producing concrete of a desired quality and at a minimum expense was considered important. The maintenance of quality was under the control of inspectors. Success in the production of concrete of a desired quality is dependent on

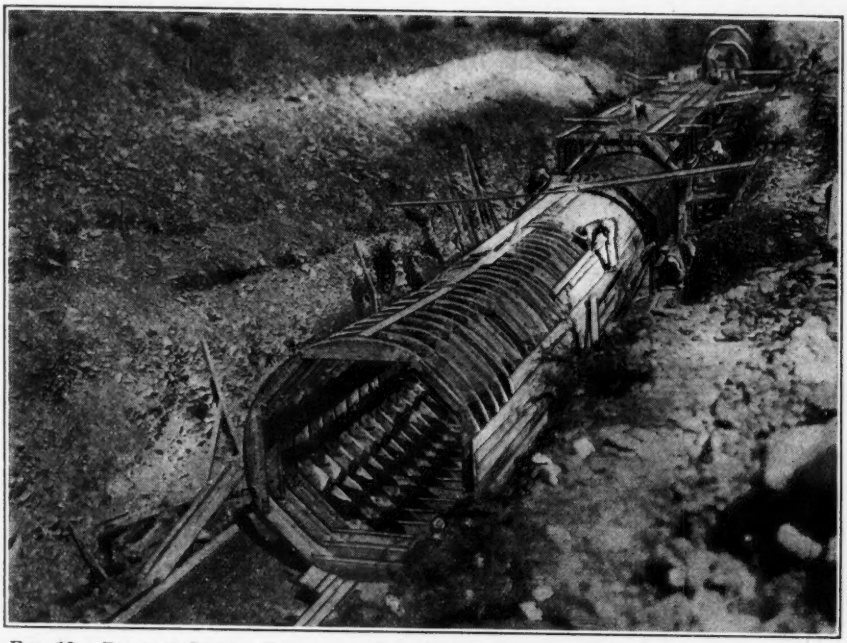

FIG. 19.—PART OF INSIDE FORMS FOR DRY CREEK SIPHON IN PLACE AND SUPPORTED ON INSIDE FORM SUPPORTS.

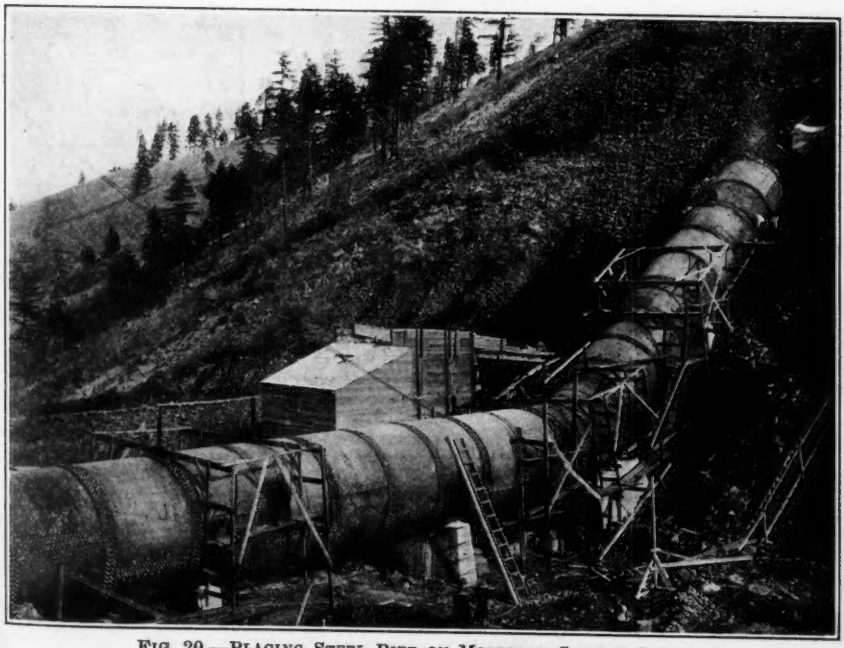

FIG. 20.—PLACING STEEL PIPE ON MORRISON CANYON SIPHON

the proper performance of all operations from the time the materials are selected and the mixture is designed until the concrete is placed and adequately cured. Failure to perform or control any individual operation properly is likely to result in a product that is non-uniform in character and either inferior in quality or wasteful in cement.

The desire of the engineers in charge to produce concrete of a uniform quality led to the incorporation of all the directions and tables to be used by inspectors in a small blue-print pamphlet entitled, "Instructions for

TABLE 4.—CONCRETE CONTROL FOR THE CONSTRUCTION SEASON, 1928; BASED ON COMPRESSION STRENGTH OF TEST CYLINDERS

Item No.	Structure	Number of cylinders	Average compressive strength (28 days), in pounds per square inch	Average water-cement-ratio	Percentages of Cylinders Tested That Had Compressive Strength Equal to: Within 10% of average strength	Within 15% of average strength	Within 20% of average strength	More than 120% of average strength	Less than 80% of average strength
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	MIX: 1: 2.4: 3.6 (DRY RODDED)								
1	Morrison Siphon Wasteway	40	3 890	0.88	52	65	75	12.5	12.5
2	Wasteway at Station 1146+30	42	3 974	0.89	53	67	79	11	10
3	Rocky Point Tunnel	50	4 334	0.87	38	68	80	8	12
4	Tunnel No. 1 on the North Branch Canal	69	3 312	0.88	38	52	73	10	17
5	Tunnel No. 1 on the South Branch Canal	60	4 138	0.87	55	75	86	7	7
6	Miscellaneous small canal lining structure (by different contractors)	145	3 848	0.90	48	65	78	12	10
7	Division 1	59	4 359	0.77	65	87	90	5	5
8	" 2	91	4 914	0.82	62	82	98	0	2
9	" 2	72	4 572	0.83	53	68	92	4	4
10	" 3	155	4 466	0.84	51	74	85	8	7
11	" 3	*90	3 755	0.84	47	74	87	6	7
12	" 3, Combination Section	*86	3 447	0.88	55	79	91	3	6
13	" 3, Combination Section	117	4 091	0.88	51	68	82	9	9
	MIX: 1: 2.0: 3.25 (DRY RODDED)								
14	Horseshoe Siphon	23	4 546	0.79	65	74	91	0	9
15	Little Creek Siphon	35	5 291	0.76	66	91	94	6	0
16	Spexarth Creek Siphon	30	4 381	0.81	70	73	90	10	0
17	Peterson Creek Siphon	44	4 271	0.81	57	77	82	13	5
18	Morrison Siphon	30	3 898	0.82	17	37	54	23	23
19	Sheep Dip Siphon	46	4 578	0.74	54	78	87	4	9
20	Little Dry Creek	30	4 515	0.83	67	80	90	3	7
21	Dry Creek Siphon	118	4 742	0.84	57	73	88	7	5
22	Siphon at Station 91+84, South Branch Canal	17	4 384	0.82	65	65	76	12	12
23	Big Creek Siphon	26	4 864	0.72	54	69	73	8	19
	MIX: 1: 1.7: 2.8 (DRY RODDED)								
24	Cle Elum Creek Siphon	17	5 444	0.72	82	88	100	0	0
	MIX: 1: 1.6: 2.9 (DRY RODDED)								
25	Horseshoe Siphon	19	5 075	0.68	42	63	74	10	16
26	Sheep Dip Siphon	7	5 150	0.64	43	86	86	0	14
	MIX: 1: 1.6: 2.4 (DRY RODDED)								
27	Pipe plant	52	5 704	0.73	58	69	77	6	17

* Sprayed with "process" after removal of mould, no damp sand curing.

Concrete Inspection and Control." Information in this section of the paper was secured from that pamphlet. Its use is not an attempt to supplant the employment of experienced inspectors, but to standardize the method of control. The method of control is the same as that advocated by the Portland Cement Association, but adapted to local conditions. Table 4 gives the results of strength tests on cylinders taken from the various structures on the project during 1928.

Aggregate.—Very complete investigations were made of local sources for sand and gravel, but it was decided to secure the material from a company that had pits near Steilacoom, Wash., on Puget Sound. These aggregates are washed with salt water; the average fineness modulus of the 1¼-in. gravel was 7.15; and the average fineness modulus of the sand was 3.08. The grading of aggregates for the dam and the reinforced concrete pipe differs from these figures.

Concrete.—In the determination of the proper mixes, extensive laboratory tests were made of the aggregates. The assigned proportions were slightly revised to meet the field conditions as the work progressed. The properties required for the concrete were high strength, high impermeability, and good workability. All these qualities were considered equally important. In most of the structures, high strength was an absolute necessity. In siphon structures subjected to high heads, the concrete must be of a known high strength and furthermore must be highly impermeable. In order to be worked around the barrel and between crowded steel bars, the concrete must also be very workable. The mixes as designed had these characteristics, as demonstrated in the field and by laboratory tests with actual specimens under compression and under hydrostatic pressure.

Method of Supervision.—An assumed strength, impermeability, and workability having been decided, and the proper mix to gain these characteristics fixed by test, the problem was to obtain the same results in the field as in the laboratory. On canal lining, for which the assigned mix is 1: 2.4: 3.6 (dry rodded) the percentages of moisture in the sand and gravel, for example, are assumed to be 3.1 and 0.1, respectively.

By consulting the Reclamation Service Instruction Book, previously mentioned, the inspector would find Table (*a*), the proper field mix, proportioned by volume (moist loose, taking into account the bulking of sand); Table (*b*), surface moisture in gravel (per sack of cement); Table (*c*), surface moisture in sand (per sack of cement); Table (*d*), batch weight for sand and gravel per sack of cement (including moisture); and Table (*e*), water-cement-ratios for varying quantities of water used per sack of cement. The amount of sand bulking for different percentages of moisture in the sand is also given; no allowance is made for bulking because of moisture content in gravel.

A complete set of these tables is given in the pamphlet for all the assigned mixes used on the project. By their use, it is a simple matter for the inspector to calculate the water to be added to secure the specified water-cement-ratio, because the number of gallons of water per sack of cement

contained in the sand and gravel can be taken from Tables (*b*) and (*c*), the weight of sand per sack of cement to be used is found in Table (*d*), and the volume of gravel is taken from Table (*a*), or if proportioning by weight, the weight of gravel may be taken from Table (*d*). The number of gallons of water to be added, the weight of sand and the volume or weight of gravel to be used being known, a trial batch is mixed. A slump test is made and more or less water is added to hold the slump below 2 in. which is the maximum allowable for canal lining. No change in the quantity of water or of the weight of aggregate need be made until a noticeable change is made in the slump, due to more or less moisture in the sand.

Inspectors' Reports.—Form 1 is a sample of a daily concrete report. Each inspector was required to make a report for each structure poured or for the work of each shift. Form 2 is a sample of the report made for each cylinder or group of cylinders. One cylinder was made from the work of each shift. When a patented paint coating method of curing was used two cylinders were made, one treated with the "Process" and one cured in damp sand. All inspectors' reports were checked in the Division offices the day after they were made, and any mistakes corrected and brought to the attention of the inspector. The reports were then forwarded to the project testing laboratory. Cylinders were collected on the same day each week. In this way a very close check was kept of the quality of the concrete that was being poured under each inspector. Any cylinders that tested abnormally high or low were brought to the attention of the inspector, and he was required to make an explanation of the cause. The standard 6 by 12-in. cylinder was used.

The type of report used in the 1928 construction season was improved for the 1929 season. The cylinder report and the concrete report were combined on one 5 by 8-in. card as shown in Form 3. This type of report was found to be better in some respects than Forms 1 and 2, which were mimeographed blanks on separate sheets of 8½ by 11-in. letter paper. The information was more compact and the filing was easier.

Use of Admixture.—It is to be noted that no allowance is made for admixture when computing the water-cement-ratio. The usual practice in this respect is to allow 200% absorption, that is, when 3% admixture is used, it is assumed that 6 lb. of water per sack of cement will be taken up by the admixture and is not available for mixing with the cement. In the design of mixes this allowance was not made; thus, there is no difference in the field conditions and the laboratory design. To disregard this feature simplifies the calculations and control of field mixing. The only objection is encountered in comparison of the concrete strength for a certain combined fineness modulus and water-cement-ratio on this job with that secured with the same values on any other job. A ratio of $W = 0.90$, computed as described herein, would become approximately 0.80 if the admixture were assumed to absorb 200% of its weight of water. In canal lining and in small and low-head structures 3% of admixture is used. In the parts of siphon barrels under heads of less than 100 ft., 1½% is used; for heads greater than 100 ft., no admixture is used. In the rich mixtures in the lower part of the high-head siphons, it has been found that the addition of admixture is a detriment.

FORM 1.—Sample Inspector's Daily Concrete Report

United States

Department of the Interior

Bureau of Reclamation

Kittitas Division—Yakima Project

Inspector's Daily Report (Concrete)

Contractor: Blank Construction Co. : Day Shift: 10/14 192 8

Main Canal, Div. No. 3 : Feature lining : Sta. 1359+20 to 1360+07

Weather Cloudy Warm — 65°

Instrument work performed None

Delays: 1 hour due to making water connections for mixer

Mixer: Type Ransome 13E ; Size 3 sk ; Ran 6 Hrs.; 86 Batches

Cement: Brand Spokane , sax per batch 3 , sax used 258

Admixture: Kind Diatomite ; % by Wgt. Cement 3 %; Lbs. used 774

Mix: Designed 1:2.4:3.6 { Dry / Rodded } ; field 1:3.14:3.77 { Moist / Loose } ; Slump 2 in.

Sand: Source Pioneer ; 3.1 % moisture; 25 % bulk; 816 lbs. per batch

Gravel: Source Pioneer ; 0.1 % moisture; 1–1/4 in. size; 11.31 C. F. per batch

Water: In sand 2.94 gals. per batch; in gravel 0.15 gals. per batch.

15.81 gals. added; 18.90 gals. total per batch; W/C ratio 84

Record of Curing Concrete

Structure	Location	Curing		Water Application		
		Begun	Ended	Method	Applic. per day	Kind of covering
L/MC	1359+20 1360+67	10/14/28		Hunt Process	Once	3 hr. after finishing

John Doe (Inspector)

Note: Use back of sheet to report inspection on placing of forms or reinforcing steel, or for supplying other pertinent information.

FORM 2.—SAMPLE CYLINDER REPORT

United States
Department of the Interior
Bureau of Reclamation
Kittitas Division—Yakima Project

Record of Concrete Test Cylinders (6" x 12") from Field

Structure: Kind L/MC; Location 1359+20—1360+07

Source of concrete for cylinders 1360+00

Cylinders made: Number of 2; Date and hour 10/14/28—9 A. M.

Mixer: Name and type Ransome 13E; Size 3 sk

Cement: Brand Spokane; Sax per batch 3

Admixture: Kind Diatomite; % by wgt. cement 3 %.

Aggregates: Source Pioneer; Size of course agg. 1-1/4 in

Water: Source Yakima River

Mix: 1:2.4:3.6 {Dry / Rodded} 1:2.51:3.77 {Dry / Loose} 1:3.14:3.77 {Moist / Loose}

Aggregate used per sack cement: Sand 272 Lbs.; Gravel 3.77 C. F.

Time of mixing 1-1/4 min.; Workability Good

Moisture in sand: 3.1%; 0.98 gals. per sack; 2.94 gals. per batch

Moisture in gravel: 0.1%; 0.05 gals. per sack; 0.15 gals. per batch

Water added: 5.27 gals. per sack; 15.81 gals. per batch

Total water in mix: 6.30 gals. per sack; 18.90 gals. per batch

Water-cement ratio .84; Measured slump 2 inches

Weather and temperature Cloudy 65°

Nature and extent of curing cylinders One Hunt Process. One 7 days in moist sand

Remarks:

Signed: John Doe (Inspector) R. Roe (Div. Engr.)

Nature and extent of laboratory curing.....

Compressive tests: Date.....; Age.....; .....lbs.;.....lbs./sq. in.

Date.....; Age.....; .....lbs.;.....lbs./sq. in.

Date.....; Age.....; .....lbs.;.....lbs./sq. in.

Broken by.....Reviewed by.....

Moisture Determination in Sand and Gravel.—Both the displacement, and the burning, methods of determining moisture in sand and gravel were used. The latter was utilized on the smaller structures where the required equipment was not at hand for the former.

U. S. DEPARTMENT OF THE INTERIOR—BUREAU OF RECLAMATION—KITTITAS DIVISION—YAKIMA PROJECT 7–897 (March, 1929)

CONCRETE INSPECTOR'S DAILY REPORT Date .....

Contractor ..... Shift ..... Canal ..... Sta. ..... to ..... Structure .....

Mixer: Name ..... Size ..... Sacks per batch ..... Brand cement ..... Sacks used ..... Diatomite ..... % Lbs. used .....

Source of water ..... Source of aggregates ..... Max. size aggr. ..... Mix used .....

Curing period ..... days ..... to ..... Method ..... Covering ..... Applications per day .....

DAY'S OPERATION RECORD—PER SACK OF CEMENT

Hour	Sand Wt. or Vol.	% of Moist.	Gals. in Sand	Gravel Wt. or Vol.	% of Moist.	Gals. in Gravel	Gals. H_2O Added	Total Gals. Water	W/C Ratio	Slump	Cyl. No.	Temp. Conc	Location in Structure

Weather, temperature, etc. .....

....., *Inspector.* Checked ....., *Office Engr.* ....., *Division Engr.*

NOTE.—Use back of card for remarks.

LABORATORY RECORD

Cyl. No.	7-day		28-day		Total Load	Load per Sq. In.	Tested by—	Report Date	By—	Attention
	Date	Vet	Date	Vet						

U. S. GOVERNMENT PRINTING OFFICE 1929

FORM 3.—CONCRETE INSPECTOR'S REPORT USED DURING THE 1929 SEASON

The burning method was used on gravel in all cases. Each inspector was furnished with a pair of balances and an alcohol flask. Each sample was burned with alcohol two or three times, or until all the moisture had been expelled. The prescribed weight of damp material for each analysis was 500 grammes. A table was given showing the percentage of moisture opposite dry weights, assuming a damp weight of 500 grammes. The values in this table were derived by allowing 0.5% for absorbed moisture in the aggregate.

At the sand and gravel bunkers the displacement method of moisture determination was adopted for sand, and the burning method for gravel. Two types of apparatus were used in the displacement method. One consisted of a 3 by 27-in. cylindrical, brass tube container with a water-gauge glass attachment. This tube would accommodate a 2 000-gramme sample of sand or gravel. It was calibrated in cubic centimeters within the range of the gauge glass and a scale graduated to 5 cu. cm. was placed directly behind the glass. The other apparatus was a graduated glass container 18 in. high, designed to accommodate a 500-gramme sample of sand. The flask had a lower and upper chamber or bulb connected by a short tubular section called the lower neck. The upper bulb gradually tapered to a uniform tube about 6 to 8 in. long, called the upper neck. The 200-cu. cm. point was marked on the lower neck, and the upper neck was marked in cubic centimeters throughout that portion used in obtaining the necessary readings for testing sand samples. Tables

were given for each type of apparatus showing the percentage of moisture opposite the flask or tube reading, assuming a 500-gramme, damp weight. These tables were compiled by determining the difference in the quantity of water displaced by dry and by damp samples. The volume of water displaced by the dry sand was assumed to be constant as long as the grading, unit weight, or specific gravity of the sand did not change materially.

Removal of Forms.—When the concrete had been poured under normal conditions the forms were required to remain for various periods (days), as follows:

	Days.
Diversion Dam:	
Mass concrete	2 to 3
Thin walls	5 to 7
Tunnels and other openings	14
Transitions, Flumes, and Combined Sections:	
Vertical walls and steel slopes	3 to 4
Flat slopes	2 to 3
Siphon Barrels:	
Outside supports and lagging	3 to 4
Inside diagonal braces	3
Inside horizontal braces	7
Inside vertical braces, arch rings, and lagging	14
Tunnel Lining:	
Tunnels	3 to 4
Beams and Slabs:	
Beams and slabs (subject to flexural stress)	21 to 28
Bridges:	
Piers	3 to 4
Abutments	5 to 7
Turnouts, Culverts, and Ditch Crossings:	
Intake or outlet	3 to 4
Outside of barrel, and cut-off collars	3 to 4
Inside of barrel	14
Lateral Drains:	
(*a*) If puddled immediately	2
(*b*) Otherwise	3 to 4

When concrete was poured at low temperatures the forms were moved at the discretion of the Division Engineer.

Curing.—It has been stated, and apparently with considerable justification, that improper curing is responsible for more defective or short-lived concrete than any other single element in its manufacture. Although curing has had a place in concrete specifications for years past, it is only in recent years that its high relative importance has become generally known, or sufficiently appreciated by construction supervisors to command whole-hearted observance of the specification provisions.

Water can make or ruin concrete. If used too freely in the mixing precess, it weakens the concrete by thinning the cementing fluid. A similar injury results if, after the cement has taken its final set, its affinity for water to effect hydration is not satisfied. Inasmuch as the plastic concrete contains sufficient water for complete hydration of the cement, the process of curing

resolves itself into one of retaining the original mixing water, or of supplying additional moisture to counteract evaporation and other losses.

Compressive strength tests of cylinders that were cured under damp sand or were permitted to dry out, indicate increases in strength for sand-cured specimens of about 75% at 10 days, 100% at 14 days, 125% at 28 days, and 145% at 4 months.

The resistance of concrete to abrasive action is similarly increased by moisture curing, since the strength relation has been found to be a reliable index of the efficiency of concrete to withstand wear.

The effect of proper curing in building up the internal structure of the concrete and making it less pervious to water is very marked. Experiments in the laboratory at Ellensburg, have shown that properly cured concrete, made of 1 part cement and 9 parts aggregate, is far less pervious than a rich uncured concrete made of 1 part cement and $3\frac{1}{2}$ parts aggregate, the rich concrete being allowed to dry out, with forms removed, in 24 hours. The importance of proper curing is manifest when it is realized that impermeability, which depends on effective curing, is now regarded as concrete's most effective defense against the forces leading to disintegration, such as frost action, leaching, alternate wetting and drying, and contact with acid or alkali-bearing soil or water.

For best results the curing process should not be interrupted. This is particularly true during the early stage of hardening, because when the interior is moist and swollen, rapid surface drying induces hair cracks (known as crazing) which may subsequently develop into structural cracks. Curing, therefore, should be started as soon as possible, and the concrete should not be permitted to dry out before it is adequately cured.

All concrete, except that cured by patented process, was kept continuously wet for at least 14 days after being poured. Except in cases where small structures could be readily covered with moist earth, the concrete was cured under wet burlap. In the case of canal lining, the first half (7 days) of the curing period was under burlap. The barrels of the concrete siphons were cured inside and outside for a period of 28 days, the outside under burlap. The cost of water-curing the canal lining was stated by one contractor to be approximately \$0.15 per sq. yd. The cost of the "process" curing averaged about \$0.135 per sq. yd.

Construction Costs

There is considerable difference in the unit bids received from various contractors. This condition is not common to this project alone. It is difficult to understand why one contractor should receive 100% more payment per unit than another operating on the same work and under the same conditions. It indicates that the competitive system of bidding is at fault.

Table 5 gives a list of unit prices as paid contractors. No attempt has been made to select either the extreme high or the extreme low, but the selections were made at random from the thirty-six contractors operating on the project. Only the more important items are given. On this project the

Government furnished all material that became a part of the completed construction work. The contractor furnished all lumber, nails, wire, etc., used in form work.

TABLE 5.—RANDOM SELECTION OF UNIT PRICES PAID TO CONTRACTORS

Description	Unit	Contractors										
		Main, North and South Branch Canal							Laterals			
		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9	No. 10	No. 11
Clearing	Acre	175.00	.....	90.00	150.00	75.00	.....	.....	.....	.....	.....	.....
Grubbing	Acre	250.00	.....	150.00	125.00	100.00	.....	.....	.....	.....	.....	.....
Canal Excavation:												
Class 1	Cubic yard	0.16	0.14	0.15	0.17½	0.14	0.15	0.25	0.35	0.29	0.29	0.39
Class 2	Cubic yard	0.55	0.32	0.30	0.42	0.28	0.36	0.50	0.90	0.75	0.60	0.70
Class 3	Cubic yard	1.50	1.25	1.25	1.50	1.50	1.25	1.70	3.00	2.00	2.00	1.00
Compacting embankment	Cubic yard	0.40	0.20	0.40	0.20	0.30	0.60	.....	.....	.....	.....	.....
Canal Lining:												
Trimming	Square yard	0.18	0.20	0.20	0.15	0.20	0.25	.....	.....	.....	.....	.....
Concrete	Cubic yard	8.00	15.00	11.50	8.00	12.00	11.50	.....	.....	.....	.....	.....
Reinforcing	Pound	0.02	0.04	0.014	0.02	0.02	0.015	.....	.....	.....	.....	.....
Structures:												
Concrete	Cubic yard	24.00	20.00	16.50	15.00	14.00	19.00	14.00	17.50	30.00	22.50	15.00
Reinforcing	Pound	0.02	0.04	0.015	0.02	0.02	0.015	0.03	0.035	0.03	0.04	0.03
Excavation:												
Class 1	Cubic yard	0.75	0.50	0.60	0.55	0.50	0.50	.....	1.25	0.75	1.25	0.60
Class 2	Cubic yard	1.50	0.80	1.00	1.00	1.00	1.00	.....	1.75	1.00	2.50	0.80
Class 3	Cubic yard	.....	3.00	3.00	.....	3.00	3.00	.....	3.50	.....	.....	1.25
Back-fill	Cubic yard	0.50	0.50	0.65	0.33	0.25	0.40	1.00	0.75	0.50	0.75	0.45
Puddle back-fill	Cubic yard	0.50	1.00	0.40	0.50	0.60	0.60	0.50	.....	.....	.....	.....
Erecting timber in bridges	1 000 ft. board measure	30.00	30.00	28.00	.....	16.00	25.00	.....	40.00	15.00	.....	30.00
Tunnel Excavation:												
11.9 diameter, Horseshoe Section	Linear feet	.....	.....	.....	.....	.....	.....	24.00	.....	.....	.....	.....
6.25 diameter, Horseshoe Section	Linear feet	.....	.....	.....	.....	.....	.....	12.50	.....	.....	.....	.....
Concrete in tunnel lining	Cubic yard	.....	.....	.....	.....	.....	.....	9.00	.....	.....	.....	.....

The cost to the contractors of common labor ranged from $4.00 to $5.00 per 8-hour day, probably the more common price being $4.50. The approximate prices paid to the skilled labor are as follows: Shovel runners, $1.43 per hour; foremen, $1.00 per hour; carpenters, $0.75 per hour; and concrete finishers (in canal lining), $1.00 per hour. Other labor was paid a proportionate price.

GENERAL CONCLUSIONS

Probably the most noticeable difference in the construction work on this project and other projects that have been built by the U. S. Bureau of Reclamation was the use of more scientific methods in the control of concrete manufacture. There was a distinct improvement in the quality and uniformity of the concrete over that poured by the more common method which was used on some of the earlier concrete on these projects. This was shown principally by appearances and the strength of cylinders and samples taken from the cured concrete, and by leakage tests made on siphons. Many engineers have a deep-seated conviction that scientific concrete control is a laboratory procedure alone. In the application of the method of concrete control used on this project, the inspectors and others engaged in directing the operations, were instructed as to the many influences affecting concrete in the

making and the necessity for co-ordinating and controlling the various detail processes if concrete of uniformly high quality was to be produced. There is an apparent increase in cost incident to closer control, but a more complete analysis will show a probable decrease in cost for the same quality concrete, due to a more careful check on the cement and aggregate quantities. It is sometimes difficult to convince a practical man on concrete work, who has had some experience, that scientific control has any advantage over the old method.

A study of Table 4 reveals some interesting facts. There were 145 cylinders made from small structures. As a general rule, these were 1 or 2 cylinders per structure. The probable number of structures from which they were taken was about 100. Their average strength was 3 848 lb. per sq. in. and 78% were within 20% of that value. They were made by thirty to forty different inspectors under temperature conditions ranging from nearly freezing to more than 100° Fahr.

The writer is indebted to the engineers of the U. S. Bureau of Reclamation at the Main Office in Denver, Colo., the Project Office, at Ellensburg, Wash., and the Division Offices at South Cle Elum and Thorp, Wash., for kind assistance in the preparation of this paper. This included the contribution of much of the information and material and the reading and correction of the manuscript.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SOME ASPECTS OF WATER CONSERVATION

Discussion

BY MESSRS. A. J. GILARDI, AND R. A. SUTHERLAND

A. J. GILARDI,[9] ASSOC. M. AM. SOC. C. E. (by letter).[9a]—The method of comparing and classifying reservoirs, dam sites, and the various types of dams described in this paper, is very useful, because, to a certain extent, it makes possible the visualization of their respective merits in comparison with such standards or "yardsticks" as those with which the hydraulic engineer may be most familiar.

Probably, the constant use of the charts given in the paper would overcome some difficulties which the writer has experienced in attempting to apply a few of them in his studies of dams. It would seem possible, however, to introduce a few simplifications and thus increase the facility and accuracy of application of the charts on the part of engineers.

In the course of his work of making numerous preliminary investigations of dam sites and reservoirs, the writer has prepared two simple and helpful charts to determine the approximate volume of dams for a wide range of sizes and types.

Fig. 28 gives directly, as abscissas, the total volumes of concrete required for gravity dams of a given height, $H_{max.}$, and for a given distance between abutments, measured at the water surface. This diagram is based on the following assumptions:

(*a*) That the dam has a gravity section which will be designated as the standard section, *C* (see Fig. 28), and is designed for uplift of 100% on 50% of the area, or what is usually called 50% uplift. The weight of the concrete has been assumed at 150 lb. per cu. ft. The section has a top width of 15 ft. and a free-board of 10 ft., which would correspond to about the same volume as a top width of 20 ft. and a free-board of only 5 ft.

(*b*) That the actual dam site has such a shape as to be replaceable by a theoretical site of the parabolic or "hill" type. This assumption is similar to that suggested by Mr. Sutherland, with the exception that two modifications of a practical nature are introduced. The first is

NOTE.—The paper by R. A. Sutherland, Assoc. M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: January, 1931, by Messrs. E. C. Eaton and A. W. Simonds; February, 1931, by Messrs. W. H. R. Nimmo and C. R. Olberg; and March, 1931, by John W. Pritchett, M. Am. Soc. C. E.

[9] Cons. and Civ. Engr., Seattle, Wash.

[9a] Received by the Secretary June 22, 1931.

that the vertex of the parabola is at the deepest part of the excavation (estimated or actual, as the case may be), the vertical distance from this point to the water surface being designated as the height of the dam, $H_{max.}$. The second assumption is that the foundation of the dam or excavation line is assumed to follow a parabolic line and end at the abutments with a depth of excavation of 5% of $H_{max.}$.

Fig. 28 covers a wide range of sizes, from heights of 50 to 800 ft. and lengths of 100 to 5 000 ft.

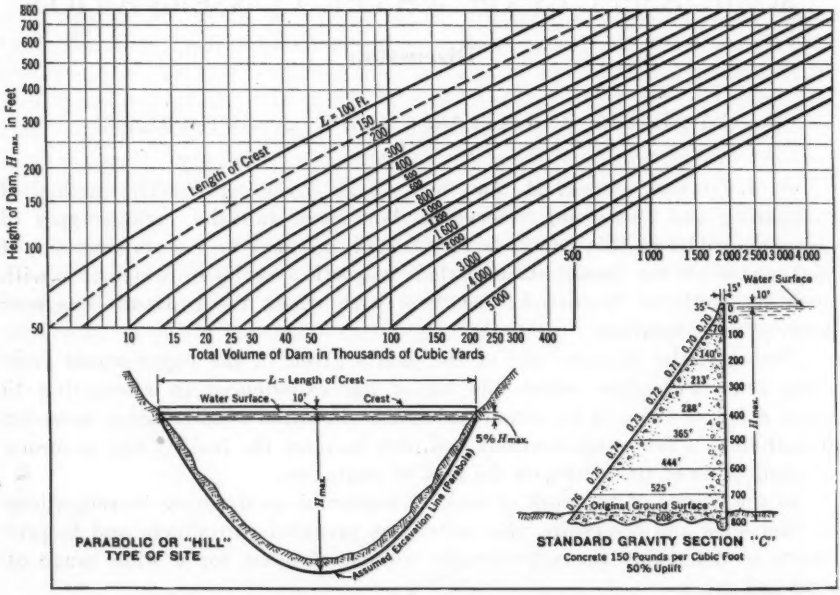

FIG. 28.—TOTAL VOLUME OF GRAVITY DAMS FOR STANDARD SECTION, *C*, AND PARABOLIC OR HILL TYPE OF SITE

Fig. 29 is intended for the purpose of modifying the volumes obtained from Fig. 28, whenever it is necessary to take into consideration sections and sites other than those used as a basis for that chart. This desirable result is arrived at by the introduction of factors, which will be designated as "section factors" and "site factors" (see Table 10).

TABLE 10.—SECTION AND SITE FACTORS FOR USE WITH FIG. 29.

Height, $H_{max.}$, in feet	Site Factors			Section Factors			
	Hill type, standard	V-type	Gorge type	*A*, average arch	*B*, no uplift	*C*, 50% uplift (standard)	*D*, 100% uplift
50	1.00	0.76	0.58	0.50	0.96	1.00	1.04
100	1.00	0.71	0.49	0.50	0.94	1.00	1.05
200	1.00	0.67	0.43	0.50	0.93	1.00	1.11
300	1.00	0.66	0.42	0.50	0.92	1.00	1.15
400	1.00	0.65	0.41	0.50	0.91	1.00	1.17
600	1.00	0.64	0.40	0.50	0.90	1.00	1.18

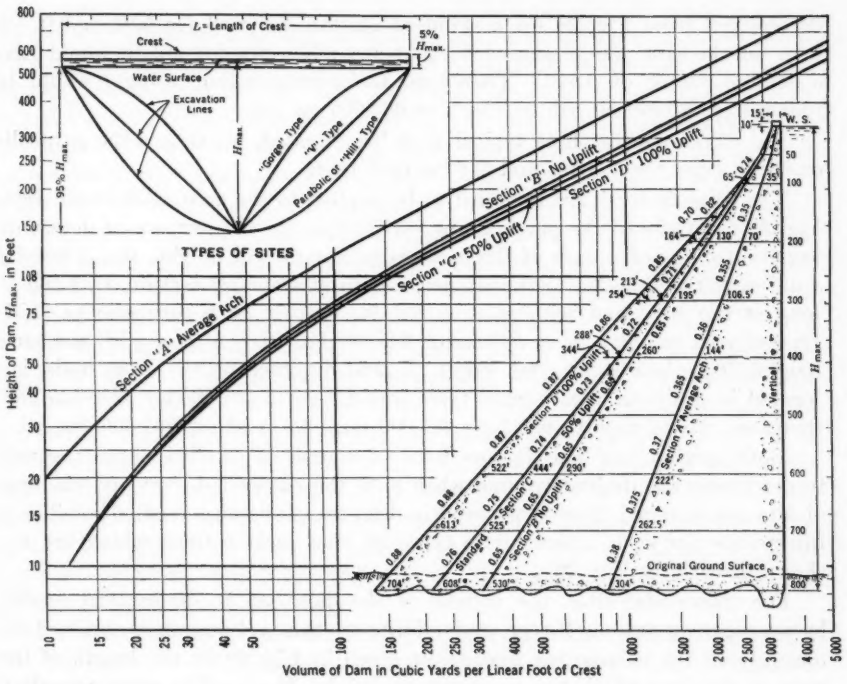

FIG. 29.

The four typical sections of dams used in Fig. 29 are the following:

Section *A*, an average arch section for which the top width is reduced to 7½ ft. with a 10-ft. free-board;

Section *B*, a gravity section designed without any allowance for uplift;

Section *C*, a gravity section which has been designated as the standard section and is designed for 100% uplift on 50% of the base, or what is generally called 50% uplift; and,

Section *D*, an ultra-conservative gravity section designed for 100% uplift on 100% of the base.

For the sake of simplicity, all the sections used have a vertical up-stream face throughout and have practically a triangular shape. While it is probable that for the higher dams the triangular section would be replaced by a more complicated section, this would mean essentially that the masses would be distributed somewhat differently, although approximately the same total volume would be required.

The "section factors" in Table 10 are intended to be applied to the volume obtained from Fig. 28 for the standard section, *C*, and the given $H_{max.}$, and length of crest, for the purpose of allowing for a change in section, such as the engineer may consider desirable in his particular case. For instance, a dam, 250 ft. high ($H_{max.} = 250$ ft.) with a crest length of 1 500 ft., on a site of the parabolic or "hill" type, would have a total volume of 685 000 cu. yd., as

determined from Fig. 28 for a standard section designed for 50% uplift. A dam for the same site, designed without any allowance for uplift, would have a section factor of 0.925. Therefore, the corresponding volume would be reduced to 685 000 cu. yd. by 0.925, or 633 000 cu. yd.

Fig. 29 shows the three typical sites investigated, which are the parabolic or "hill" type, the V-type, and the "gorge" type.

The "site factors" are intended to be applied to the volume obtained from Fig. 28 for a site of the parabolic or "hill" type, for the purpose of determining the volume of a dam of the same crest length and height, $H_{max.}$, but for a different type of site. For instance, a dam of standard section, *C*, 1 500 ft. long at the crest and with $H_{max.} = 250$ ft., located on a parabolic or "hill" type of site, would have a volume of 685 000 cu. yd. A dam with the same height, $H_{max.}$, and same crest length, *L*, and designed on the same basis, but located in a site of the "gorge" type, would have a site factor of 0.425 and, therefore, would require 0.425 by 685 000 cu. yd., or about 291 000 cu. yd.

Both section and site factors may of course be applied simultaneously to determine the desired volume when both the basis of design and the type of site are changed from those used in Fig. 28. It is also readily possible to interpolate for such intermediate types of sites and sections which are not shown directly in Fig. 29.

For trapezoidal sites, the volume of the dam can be determined readily by segregating into a V-type and adding to this volume that obtained by multiplying the volume per linear foot given in Fig. 29 by the length of the part of the dam which has the constant height, $H_{max.}$. The same procedure of segregation and summation may be applied to such others for which the three typical sites may not be directly applicable. The diagrams presented herewith are intended, of course, to be used for preliminary purposes only, and as approximate checks in other cases.

The writer has not had the time required to prepare a detailed tabulation to compare the volumes obtained from the diagrams with those of dams actually constructed. However, a few comparisons have shown to his satisfaction that the diagrams give results which are sufficiently accurate for the purposes for which they are intended. For instance, in a preliminary investigation of a gravity dam, straight in plan, with a crest length of 1 545 ft. and a height, $H_{max.}$, of 252 ft., a detailed analysis requiring many hours of work resulted in a volume of 680 000 cu. yd., whereas Fig. 28 gave at a glance a volume of 720 000 cu. yd. Upon investigation, it was found that the small difference of 5½% was probably due to a "hump" on the right side of the actual dam site at about mid-height, which, of course, the parabolic or "hill" type of site had not taken into consideration.

Another example for which a comparison was made, was the Gibson Dam, an arched structure with a crest length of 960 ft. and a total height of 195 ft., corresponding to an $H_{max.}$ of 185 ft. As already mentioned, the actual volume of the dam alone, as constructed, was 160 700 cu. yd. Fig. 28 gives a volume of 250 000 cu. yd. for a standard gravity section, *C*. By comparison of the section of the Gibson Dam with the average arch section, *A*, on Fig. 29, it

will readily be seen that the section of the Gibson Dam is considerably thicker at all points, the width at the top being 20 ft. against 7½ ft., and the depth, at a depth of water of 150 ft., being about 60 ft. against 50 ft. In addition, the Gibson Dam has a considerable mass of concrete added at the maximum section. In view of this, it is reasonable to interpolate between the section factors given on Fig. 29 for Sections *A* and *B* by adding 25% to the section factor for *A*. This results in a section factor of 0.625, which, in turn, determines the volume of the Gibson Dam at 0.625 by 250 000 cu. yd. = 156 200 cu. yd., or only 2.8% less than the actual volume as constructed. In using the diagrams for estimating the volume of arched dams, the developed length of the crest must be introduced.

The diagrams will sometimes be found useful in reversing the usual procedure of determining the volume from given dimensions. Assuming, for instance, that, for a given dam site, only a limited sum of money is available and that, therefore, the maximum volume of concrete is fixed, it is possible to determine in a few minutes what maximum height of dam could be constructed. The diagrams have been prepared for non-overflow sections of dams. However, they have been found quite useful also for overflow sections, with minor arithmetical adjustments. Diagrams similar to those presented herewith, could be prepared for earth dams, rock-fill dams, and other types.

In the course of the extensive computations and other work required for the preparation of the diagrams, the writer has had the valuable co-operation of Mr. L. S. Moore, of the U. S. Engineer Office, at Seattle, Wash., to whom thanks are extended herewith.

R. A. SUTHERLAND,[10] ASSOC. M. AM. SOC. C. E. (by letter).[10a]—The discussion by Mr. Eaton is particularly interesting in that it deals with problems similar in many ways to those met by the writer in Australia. The increase in flood severity and in the quantity of detritus carried, due to destruction of natural cover, as mentioned by Mr. Eaton, has become a problem of real gravity in many countries. The writer believes that in America this problem has been faced for many years by the reservation of immense areas of forest, with adequate provision for safeguard from fire. In most countries, however, the destruction of forest and other cover is allowed to go on far beyond a safe point before anything is done to prevent it.

Mr. Eaton has touched a vital point when he states that "the costs of the remaining dam sites in Los Angeles County mountain territory are relatively high." This is now becoming true for practically every country, and the increasing cost of storage has prompted the intensive study which has been given to economy in dams during the twenty years since about 1910. The writer's paper was developed and presented with the object of aiding in the study of how to obtain the greatest "value for money," having regard not only to the dam but to the reservoir itself.

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[10] Chf. Engr. and Asst. Mgr., Christmas Island Phosphate Co., Christmas Island, *via* Singapore, Strait Settlements.

[10a] Received by the Secretary September 22, 1931.

Relative to the justified expenditure on a dam, there are, as Mr. Eaton points out, many factors to consider; looking at the dam as an insurance against either an excessive or a defective flow of water, it is obvious that the premium which can be afforded for such insurance will be in proportion to the risks to be guarded against. In the case of dams built solely or mainly to create head (and these form probably a growing proportion of the total), the value of such head is another factor to be considered, whether it is used to create power or to avoid pumping for water supply.

The method of comparing dams and storages, as described by Mr. Eaton, is of great interest and value, enabling a comprehensive preliminary survey to be made of a series of sites on any one stream or on adjacent streams. The increase of unit cost with increasing capacity as shown by Fig. 22, is characteristic of storages having a relatively low value of m. The cost of most dams varies with the height according to a power of from 2.5 to more than 4, depending on the site and the type of dam, while many storages have lower values of m.

The writer is gratified that his paper has been of interest in a district which he has always admired for its progressive water administration policy.

The writer agrees fully with Mr. Simonds as to the paramount importance of the geological aspect of water conservation. It is essential that a dam should form a stable and integral part of the landscape, or to use a mechanical analogy, that it should be a "weld" and not a "patch." Scarcely less importance should be attached to the geological properties of the reservoir basin. This phase of the subject is treated more thoroughly in the comprehensive manuscript from which the material for this paper was condensed.[11]

As Mr. Simonds points out, expenditure on diamond drilling is generally not only necessary, but is a true economy. The exploration of the dam site should not stop at the examination of the rock strata, but should be sufficiently extensive to enable the history and probable stability of the entire surrounding terrain to be visualized. The writer can cite a case in which the site of a large dam to be built for power generation was most excellently prospected by drilling and tunneling, but in which, nevertheless, the completed works had to be shut down soon after commencing operation, on account of unsuspected weaknesses in the adjacent terrain, under the altered conditions. Provision against this meant a large further expenditure in remedial works, in addition to heavy losses through idle plant.

The writer regrets that Mr. Simonds considers his curves to be of doubtful practical value. The topographic survey is essential when the project has reached a certain stage, and the writer has never suggested otherwise, but he has seen thousands of dollars wasted in careful surveys of projects which were never begun. In many cases this useless expenditure could be avoided by a common-sense application of the broad principles introduced in this paper.

Mr. Nimmo's able discussion of the reasons for the characteristic properties of different types of reservoir is very interesting, and appears to be founded on a scientifically correct basis.

[11] Bound copy on file at Engineering Societies Library, New York, N. Y.

Mr. Olberg considers that the classification of reservoirs into different types is of importance only as a means of definition and comparison. The original idea of this classification is due to Mr. Nimmo, and it has been fairly well established by theoretical considerations and from actual practice. It seems desirable to be able to recognize any given reservoir as belonging to a certain type, since each type has distinct characteristics of its own as regards seepage, evaporation, and flow-regulating properties. This aspect of the subject has been merely touched on in the paper, but might be considerably elaborated.

No mathematical legerdemain is required to prove that m is the ratio of maximum depth to average depth. The average depth equals $\frac{\text{capacity}}{\text{water=spread area}} = \frac{C_w \ h_1^m}{m \ C_w \ h_1^{m-1}}$, and dividing this into the maximum depth, h_1, gives m. As regards the constant, C_w, this is the theoretical capacity at unit depth, but this fact is of no practical importance.

Mr. Olberg is correct in preferring the plotting of the logarithmic line to the use of the alignment charts (Figs. 4, 6, and 8) as a means of finding the correct constants for a reservoir or dam site. The alignment charts, however, are sufficiently accurate for practical purposes and give the desired result at a glance without calculation or the chance of arithmetical error.

Referring to the remarks of both Mr. Olberg and Mr. Nimmo regarding the use of the site-area and the site-width equations, the writer has evidently not made the fact sufficiently clear that the use of Equation (1) as the starting point in deriving the formulas for the volume of dams is purely empirical. It was found to give satisfactory results, and the use of the width of dam site was not found satisfactory. It should also be pointed out that Equation (7), derived from Equation (1), really applies not to the site itself, but to the equivalent theoretical site as shown in dotted lines in Fig. 9. If the width is measured at a large number of short-height intervals (that is, making dh very small), and the logarithmic line is plotted, it should give exactly the same result as the plotting of site areas. In practice, it is easier to let the planimeter assist in striking the average, than to plot so many points.

Mr. Olberg's assumption that Equation (7) was derived from the capacity equation for the reservoir is in error. The writer has not endeavored to derive any relationship between the capacity and site-area equations. It is doubtful whether even a quantitative relationship holds between the constants of the two equations; and examples will readily come to mind of large storages being impounded at a narrow gorge, and, conversely, of long and costly dams being required for a relatively small storage. The fact is that the properties of dam sites are frequently determined by some rather local geological condition, while the properties of reservoir basins are determined by the geological and climatological conditions on a fairly extensive terrain.

Mr. Nimmo has pointed out the existence of a compound type of reservoir, as exemplified by Lake Cushman. The writer believes this to be very excep-

tional, and has never encountered a similar case. He is glad to note that Mr. Nimmo and Mr. Olberg have found his formulas to give results sufficiently accurate for preliminary investigations. For such purposes the data available are usually not sufficient to warrant the more laborious methods frequently used in computing quantities, and the accuracy of these methods is often more fictitious than real.

The volume of dams designed by the Cain formulas or by the trial-load method, might be found as a certain percentage of the volume of the corresponding plain arch dam having a given "cylinder" stress, such percentage being determined from experience. The writer has not sufficient data to enable him to indicate a coefficient for this use. Mr. Simonds has given the actual volume of the Gibson Dam as 160 000 cu. yd. The volume computed by the writer's method is 142 000, which indicates an error of 11 per cent.

Mr. Olberg suggests that an adjustment might be made in the formulas for "specific capacity," to allow for the crest and foundation margins. The difficulty is that by altering the arguments in the numerator and denominator of the expression, the possibility of deriving a simple expression is lost. The writer made an adjustment in the coefficient to allow for the average conditions and margins stated, and the formulas will be approximately correct under those conditions, but will be increasingly in error as the dam considered is very low or very high.

The method of determining the most economical size of installation, of which Mr. Olberg would like to have more information, depends on a use of the Rippl curve, or other means of determining the storage required to provide a given regulated flow. For various values of regulated flow, various sizes of dam are required, the quantity of material in which can be found approximately from the formulas given in the paper; and, by applying proper unit costs, the total cost of the different dams can be found. Dividing these costs by the values of the regulated flow, therefore, enables the point to be found, which will give the lowest cost per unit of regulated flow.

Data for developed and proposed storage in Texas for different purposes, as given by Mr. Pritchett, are of great interest. The large value of 2 500 000 cu. ft. being developed or approved for water power is remarkable. In view of the arid nature of Texas soil, it would have been of interest to have figures showing the acre-feet of storage considered necessary per acre of irrigated land, and the writer presumes that this figure would be relatively high. It is noted from Table 9 that the computed and diagram capacities for the larger storage generally agree fairly closely. The values of m also agree reasonably well, while those of C_w show wide discrepancies. Actually, it would be found that the computed values would give an exact agreement of capacity for one value of height, with greater or smaller errors at other values of height, and the object in selecting values of m and C_w should be to spread the errors so that they are nowhere excessive.

Mr. Gilardi's very useful and practical diagrams should be of great value in investigations of the cost of dams when the leading particulars, such as height, crest length, and type have been decided. They should give prac-

tically the same results in parallel cases, as the writer's Fig. 10, for any particular case. For example, consider a site for which $n = 1.5$, $C_a = 10$, and $H_{max.} = 100$. In that case, $L = 150$. This gives, from Mr. Gilardi's Fig. 28, a volume of 15 500 cu. yd. and from Fig. 10, 15 390 cu. yd. (total height = 110 and mean $K_g = 0.73$, as shown on Fig. 28).

The use of site and section factors, applied to the results obtained for a standard gravity section, is a distinctly useful artifice, although interpolation in two senses may not be always easy. For sites of the composite type, not covered by the writer's procedure, Mr. Gilardi's proposal to divide the site into its component parts and determine the separate volumes from his two diagrams, is a practical method of procedure.

It is necessary to correct the impression that the "gorge" and "hill" types of dam site mentioned by Mr. Gilardi, have any necessary connection with the classification of reservoirs into these types adopted by the writer. The type of reservoir depends on the average of the cross-sections of an extensive stretch of river, including any tributaries in the reservoir basin. Where these cross-sections are predominatly of the gorge type, the reservoir is a gorge type. In many cases the nature of the dam site may be determined by purely local geological conditions, and it may be of a different type as, for example, the hill type. In point of fact dam sites below reservoir basins of the gorge type are frequently of the hill type, and the gorge type of site may occur below a lake type of basin. For this reason the writer did not attempt to draw any connection between the type of basin and its dam site.

Regarding the writer's statement following Table 1 that in the cases examined by him, the value of m for reservoirs occurring on the same river was a function of the elevation, Mr. Nimmo has kindly pointed out a case in which a large number of reservoir sites on one river were investigated, and in which this relationship does not hold. This is the Umpqua and North Umpqua River, in Oregon. Fig. 30 and Table 11 were plotted by Mr. Nimmo

TABLE 11.—CHARACTERISTICS OF PROPOSED RESERVOIR SITES ON THE UMPQUA AND NORTH UMPQUA RIVERS, OREGON

Dam Site No.	Mileage from Scottsburg, Ore.	Elevation, in feet	Value of C_w	Value of m
25	1	0	70.8	1.75
24	11.5	41.5	149.2	1.60
23	29.5	100	91.6	1.64
22	45	166	53.3	1.72
21	49.5	186	48.5	1.72
20	63	255	1.162	2.51
19	69	294.5	1.022	2.68
17	91	439	23.2	1.82
16	97.5	506	42.2	1.73
15	106	596	24.25	1.89
14	112	660	38.5	1.80
12	122.5	847	16.6	1.76
10	137.5	1 187	6.82	1.82
9	145.5	1 450	10.6	1.68
8	152	1 750	10.97	1.62

from data supplied by the U. S. Geological Survey.[12] While it is true that the relationship does not hold in this case, Table 11 does show in a general manner that, for reservoirs of similar type, C_w varies with the catchment. For example, compare Sites 10 and 17; 16, 21, and 22; and 8, 23, and 24. Site 14 does not conform to this generalization.

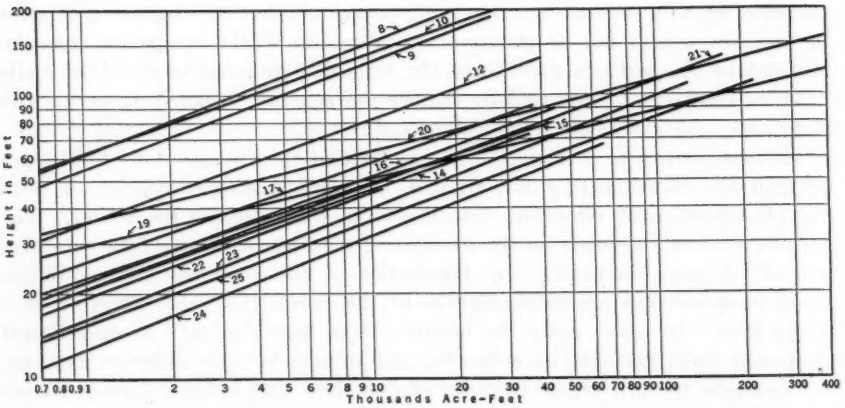

FIG. 30.—RESERVOIR CAPACITIES, UMPQUA AND NORTH UMPQUA RIVERS, OREGON.

Referring to that part of the writer's paper, headed "Arch Dams," the following should be added: The Gibson Dam (Table 7, Item 12), was designed by the trial-load method and the use of Fig. 15 is not strictly applicable. The discrepancy between the actual and chart values of W_s shows that the actual volume of concrete is much more than when computed from the stress derived from the batter of the mid-section. This increase is due to the considerable thickening of the flanks of the dam and the excess section over the theoretical triangular section.

[12] *Water Supply Paper 636*, U. S. Geological Survey.

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DISCUSSIONS

FORMULAS FOR RAINFALL INTENSITIES OF LONG DURATION

Discussion

BY MERRILL M. BERNARD, M. AM. SOC. C. E.

MERRILL M. BERNARD,[15] M. AM. SOC. C. E. (by letter).[15a]—The closing discussion will be confined to the several important points raised and to a number of interesting facts which developed in consolidating, extending, and checking the rainfall intensity charts.

Agricultural drainage and stream-flow analysis have opened a comparatively new field of use for the "rational" method, and have created the need for formulas expressing the relationship between rainfall intensity, duration, and frequency for time intervals longer than 120 min. The time of concentration for a system of storm sewers is seldom more than 180 min.; concentration time at the upper end of an agricultural drain is never less than 60 min. The two fields will seldom overlap in the same problem, and there would be little advantage in covering all duration periods in one expression. The Meyer formulas, which may be safely extended to durations of 200 min., adequately meet the needs for design in the field of storm sewers. The writer's formulas, with values of t ranging from 60 min. to several days, may be used in the design of extensive drainage systems and in the analysis of stream flow of considerable magnitude.

Factor b was not introduced into the intensity formula for the following reasons:

(1) Its influence on intensity rapidly diminishes for duration periods beyond 100 min.
(2) The refinement gained in introducing this factor is not warranted when many greater discrepancies are accepted in the course of design.
(3) The study, so extensive as to embrace one hundred and seven 2°-quadrangles, could only be undertaken by utilizing the facility of logarithmic plotting, which permitted the rapid and accurate adjustment of the curves, as straight lines, to the plotted points.

NOTE.—The paper by Merrill M. Bernard, M. Am. Soc. C. E., was published in October, 1930, *Proceedings*. Discussion on the paper has appeared in *Proceedings* as follows: February, 1931, by Messrs. W. B. Gregory, W. W. Horner, and Adolph F. Meyer; and May, 1931, by Messrs. C. E. Grunsky, R. W. Powell, and Charles W. Sherman.

[15] Cons. Civ. Engr. (Bernard Engrs., Inc.), Crowley, La.

[15a] Received by the Secretary, July 24, 1931.

In extending the limits of the charted area, the writer has followed the suggestion that n need not vary with frequency, but may be considered to be constant and need only vary with geographical location, as may the exponent, x, in the formula,

$$i = \frac{KF^x}{t^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

in which,

i = average rainfall intensity, in inches per hour.
F = frequency, in years.
t = duration of rainfall, exceeding 1 hour, in minutes.
K = a coefficient, depending for value on locality.
x and n = exponents, depending for value on locality.

The range in value of both the exponents is too great to permit further generalization by the use of average values. The formula may easily be modified if it becomes necessary to express time in hours. Rainfall depth is obtained by multiplying the computed average rainfall intensity by duration in hours.

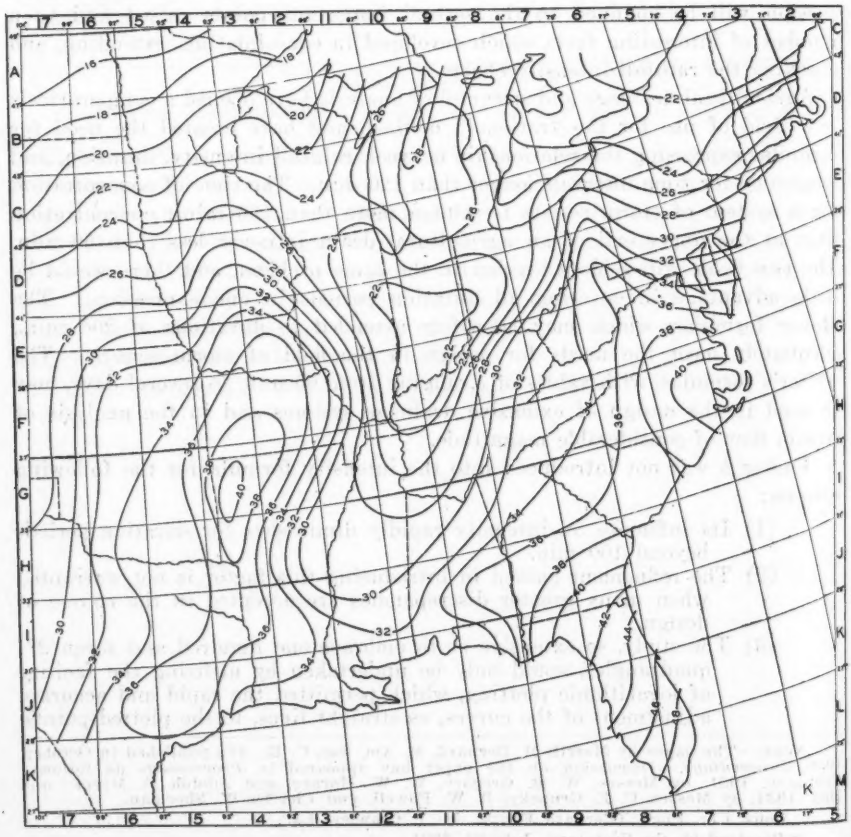

FIG. 18.—VALUES OF COEFFICIENT, K, IN EQUATION (20)

Figs. 18, 19, and 20 have been constructed similar to Figs. 4 to 15, inclusive. Their limits have been extended to embrace all that part of the United States east of the 101st Meridian, completing the area covered

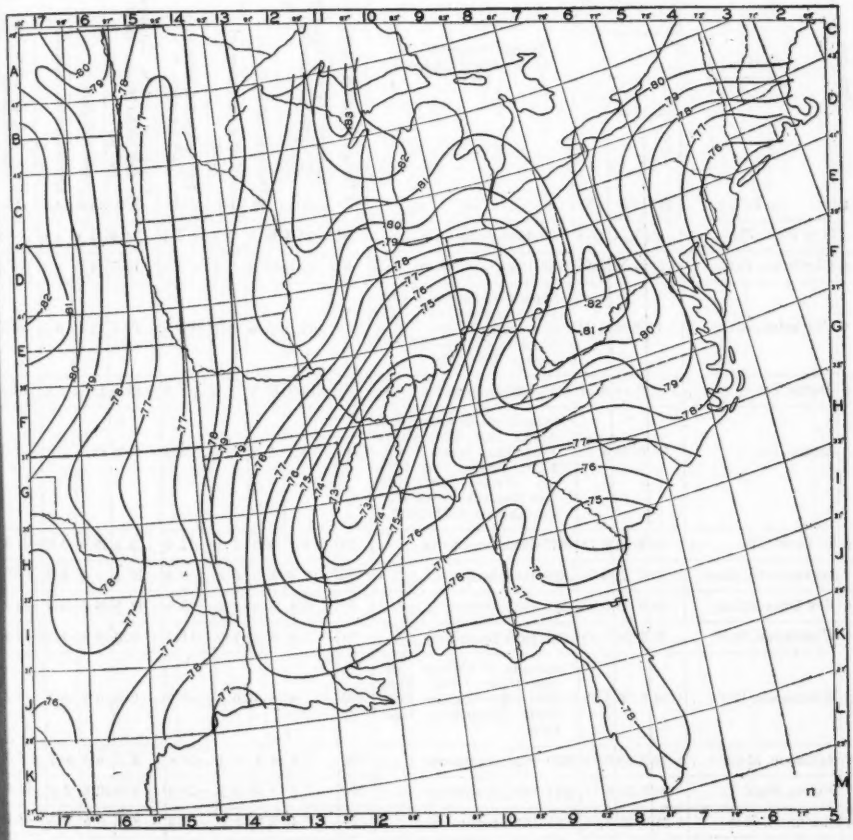

FIG. 19.—VALUES OF EXPONENT, n, IN EQUATION (20)

by the Meyer and Morgan studies. Fig. 18 gives values for the coefficient, K; Fig. 19, values for the exponent, n; and Fig. 20, values for the exponent, x, in Equation (20).

Mr. Powell has shown that, at Columbus, the 30-day rainfall depths, as computed from the charts, agree quite accurately with the observed values taken from a composite record for the locality. The writer has followed the suggestion that this be verified at other points. The results are shown in Table 17, in which thirteen well-distributed points are listed according to the magnitude of the coefficient, K, in their intensity formulas. These are followed by the computed and observed 30-day rainfall depths, the deviation

TABLE 17.—COMPARISON OF RAINFALL

Item No.	Station	Values of Coefficient and Exponents, Equation (20)			Rainfall record, U. S. Weather Bureau	Record length, in years	Rainfall Intensities for							
							F=5				F=10			
		K	x	n			Computed values, Equation (20) with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values	Computed values, Equation (20) with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	Key West, Fla.	48	0.25	0.78	1832–1930, broken	81	12.4	12.9	0.5	−3.9	14.8	15.8	1.0	−6.3
2	Charleston, S. C.	38	0.18	0.75	1832–1920, continuous	89	12.0	12.2	0.2	−1.6	13.7	14.8	1.1	−7.4
3	Charleston, S. C.	38	0.18	0.75	Composite of Pinopolis, 1900–1920; Ferguson, 1886–1920; Georgetown, 1893 – 1920; St. Johns, 1846–1861; St. George, 1894–1920	126	12.0	11.6	0.4	+3.4	13.7	13.2	0.5	+3.8
4	Austin, Tex.	36	0.24	0.77	1856–1930, continuous	73	10.1	10.9	0.8	−7.3	11.9	12.7	0.6	−4.7
5	Austin, Tex.	36	0.24	0.77	Composite of Quadrangle 16-J, Austin, 1856–1920; Blanco, 1896–1920; Boerne, 1892–1920; Cuero, 1883–1920; San Marcos, 1893–1920; New Braunsfels, 1888–1920; San Antonio, 1871–1920	330	10.1	9.8	0.3	+3.1	11.9	12.1	0.2	−1.7
6	St. Louis, Mo.	34	0.20	0.78	1837–1930, continuous	94	8.1	9.2	1.1	−12.0	9.3	10.5	1.2	−11.4
7	Leavenworth, Kans.	33	0.20	0.77	1836–1930, broken	95	8.8	8.8	0.0	0.0	10.1	9.9	0.2	+2.0
8	New Orleans, La.	33	0.34	0.78	1836–1930, broken	85	9.8	13.6	3.8	−28.0	12.5	14.5	2.0	−13.8
9	Washington, D. C.	33	0.20	0.78	1824–1930, broken	91	7.8	8.8	1.0	−11.4	9.0	10.5	1.5	−14.3
10	Washington, D. C.	33	0.20	0.78	Composite of College Park, 1861–1920; Great Falls, 1888–1920; Takoma, 1899–1920; Alexandria, 1853–1905	106	7.8	8.3	0.5	−6.0	9.0	9.2	0.2	−2.2
11	Baltimore, Md.	32	0.20	0.78	1820–1920, continuous	100	7.6	8.5	0.9	−10.6	8.7	9.4	0.7	−7.5
12	Boston, Mass.	26	0.23	0.77	1821–1920, continuous	100	7.2	9.4	2.2	−23.4	8.5	10.8	2.3	−21.3
13	Cincinnati, Ohio.	24	0.23	0.76	1835–1930, continuous	96	7.4	8.9	1.5	−16.9	8.7	9.5	0.8	−8.4
14	Columbus, Ohio.	24	0.23	0.77	Composite of Columbus, 1878–1921; Ohio State University, 1882–1921; Westerville, 1854–1902	130	6.8	7.7	0.9	−11.7	7.9	8.4	0.5	−6.0
15	St. Paul, Minn.	24	0.27	0.78	1837–1930, continuous	94	6.4	7.5	1.1	−14.7	7.7	9.2	1.5	−16.3
16	Jamestown, N. Dak.	17	0.33	0.79	Composite of Ashley, 1892–1920; Edgerly, 1901–1920; Fullerton, 1898–1920; Fort Totten, 1869–1890; Jamestown, 1892–1920; Manfred, 1903–1920; Melville, 1898–1920; Napolian, 1892–1920 New Rockford, 1893–1915	200	4.5	5.1	0.6	−11.8	5.6	6.7	1.1	−16.4
17	Average deviation as a percentage of the observed values.	....	....	....	.....	....	.....	.....	.....	10.4	.....	.....	.....	−9.3
18	Algebraic average of deviations as a percentage of the observed values.	....	....	....	.....	....	.....	.....	.....	−9.5	.....	.....	.....	−8.2

INTENSITIES FOR VARIOUS FORMULAS

VARIOUS FREQUENCIES, *F*.																		
F=15				*F*=25				*F*=50				Record length, in years	*F*=100				Average percentage deviation from observed values	Algebraic average deviation percentage of observed values
Computed values, Equation (20), with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values	Computed values, Equation (20), with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values	Computed values, Equation (20), with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values		Computed values, Equation (20), with Columns (3), (4), and (5)	Observed values	Deviation from observed values	Percentage, deviation from observed values		
(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)
16.3	16.6	0.3	−1.8	18.5	17.1	1.4	+8.2	22.0	18.2	3.8	+20.8	81	24.7	19.7	5.0	+25.4	11.1	+7.1
14.7	15.1	0.4	−2.6	16.1	16.0	0.1	+0.6	18.3	17.2	1.1	+6.4	89	20.2	19.1	1.1	+5.8	4.1	+0.2
14.7	15.1	0.4	−2.6	16.1	18.2	2.1	−11.5	18.3	21.5	3.2	−14.9	100	21.5	27.2	5.7	−21.0	9.5	−7.1
13.1	12.9	0.2	+1.7	14.8	16.2	1.4	− 8.6	17.4	20.0	2.6	−13.0	73	19.2	20.8	1.6	−7.7	7.2	−6.6
13.1	12.6	0.5	+4.0	14.8	14.8	0.0	0.0	17.4	16.2	1.2	+7.4	100	20.6	19.4	1.2	+6.2	3.7	+3.2
10.1	10.7	0.6	−5.6	11.2	11.1	0.1	+0.9	12.8	11.8	1.0	+8.5	94	14.5	17.0	2.5	−14.7	0.8	−5.7
10.9	10.6	0.3	+2.8	12.1	11.1	1.0	+9.0	13.8	12.6	1.2	+9.5	95	15.9	15.8	0.1	+0.6	4.0	+4.0
14.3	14.7	0.4	−2.7	17.0	17.0	0.0	0.0	21.5	19.5	2.0	+10.3	95	25.7	22.5	3.2	+14.2	11.5	−3.3
9.8	10.7	0.9	−8.4	10.8	11.5	0.7	−6.1	12.5	14.2	1.7	−12.0	91	14.3	14.4	0.1	−0.7	8.8	−8.8
9.8	9.6	0.2	+2.1	10.8	9.9	0.9	+9.1	12.5	10.3	2.2	+21.3	100	14.3	11.6	2.7	+23.3	10.7	+7.9
9.5	9.5	0.0	0.0	10.5	10.5	0.0	0.0	12.0	11.0	1.0	+9.1	100	13.9	12.3	1.6	+13.0	6.7	+0.7
9.3	11.1	1.8	−16.2	10.4	11.8	1.4	−11.9	12.3	12.1	0.2	+1.7	100	14.0	12.4	1.6	+12.9	14.6	−9.7
9.6	9.9	0.3	−3.0	10.7	10.5	0.2	+1.9	12.6	11.6	1.0	+8.6	96	14.8	12.0	2.8	+23.3	10.3	+0.9
8.7	9.0	0.3	−3.3	9.8	9.8	0.0	0.0	11.5	10.9	0.6	+5.5	100	13.5	12.1	1.4	+11.7	6.4	−0.6
8.6	9.5	0.9	−9.5	9.9	10.5	0.6	−5.7	11.9	11.2	0.7	+6.3	94	14.8	11.8	2.2	+18.6	11.8	−3.6
6.4	7.3	0.9	−12.3	7.6	8.0	0.4	−5.0	9.6	8.9	0.7	+7.9	100	12.0	9.3	2.7	+29.1	15.5	−1.4
.....	.....	.....	4.9	.....	.....	.....	4.9	.....	.....	.....	10.2	.....	.....	.....	.....	14.3	9.0	.....
.....	.....	.....	−3.6	.....	.....	.....	−1.2	.....	.....	.....	+5.2	.....	.....	.....	.....	+8.8	.....	−1.4

of the computed from the observed being expressed as a percentage of the observed. The study has a threefold interest:

First.—It has submitted the charts to a rather extensive check.

Second.—It indicates that the intensity and depth curves taken from the charts may be extrapolated to durations considerably beyond four days.

Third.—Considering monthly rainfall depth as a rate-duration relationship, it presents the opportunity to compare single station records of nearly 100 years, with composite records of equal or greater length.

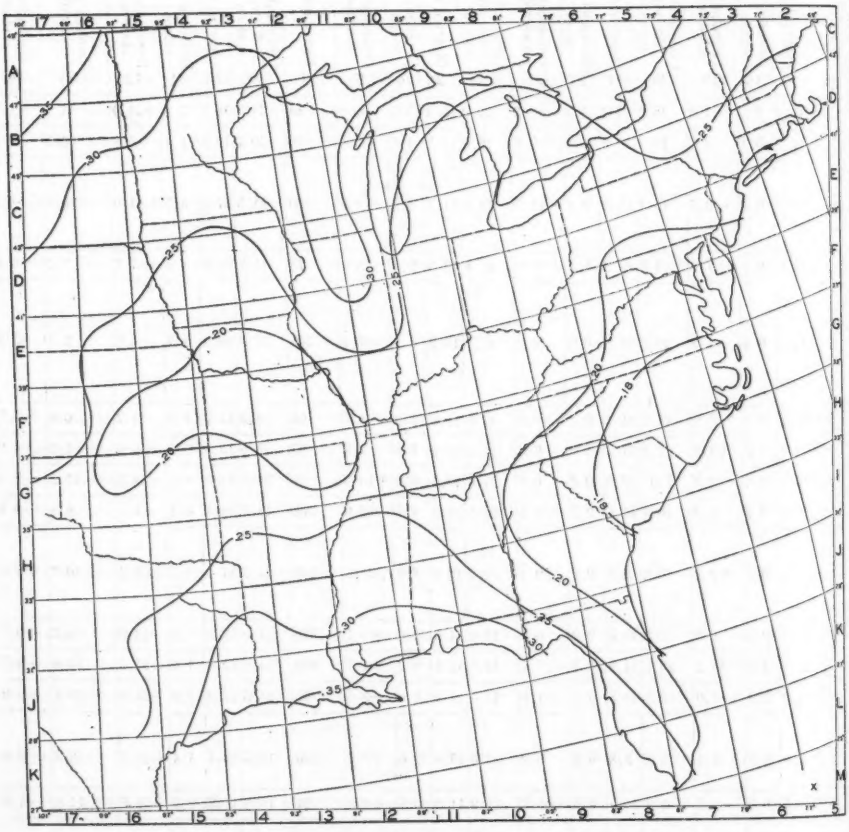

FIG. 20.—VALUES OF EXPONENT, x, IN EQUATION (20)

Because of meager intensity-duration data the theory of the composite record has not been fully established or explained, although it is rapidly being accepted as the basis for design. Composite records, varying in length, number of stations, distribution of recorded years, and arrangement of stations within the area represented, are compared with single station records at Charleston, S. C., Austin, Tex., Washington, D. C., and Columbus, Ohio, the latter being compared with the single station record at Cincinnati, Ohio.

TABLE 18.—COMPARISON OF RESULTS TAKEN FROM FIGS. 18, 19, AND 20, WITH OTHER VALUES FOR BOSTON, MASS., AND DETROIT, MICH.

Station	Frequencies, F, in years	5				10				15				25				50				100			
	Duration of rainfall, t, in minutes	60	200	500	1 000	60	200	500	1 000	60	200	500	1 000	60	200	500	1 000	60	200	500	1 000	60	200	500	1 000
Boston, Mass..	Sherman equation: $i = \frac{16\,F^{0.27}}{(t+7)^{0.70}}$	1.30	0.59	0.32	0.20	1.57	0.71	0.38	0.24	1.75	0.80	0.42	0.26	2.01	0.91	0.49	0.30	2.42	1.10	0.59	0.36	2.92	1.36	0.71	0.44
Boston, Mass..	Writer's Figs. 18, 19, and 20, equation: $i = \frac{20\,F^{0.25}}{t^{0.77}}$	1.61	0.63	0.31	0.18	1.89	0.75	0.37	0.22	2.07	0.82	0.40	0.24	2.33	0.92	0.46	0.27	2.73	1.08	0.53	0.31	3.20	1.27	0.63	0.37
Detroit, Mich.	Wagnitz and Wilcoxen equation: $i = \frac{37.6\,F^{0.233}}{(t+8)^{0.833}}$	1.56	0.60	0.28	0.16	1.86	0.72	0.34	0.19	2.08	0.80	0.37	0.21	2.33	0.92	0.43	0.24	2.85	1.10	0.51	0.28	3.43	1.32	0.61	0.34
Detroit, Mich.	Writer's Figs. 18, 19, and 20, equation: $i = \frac{25\,F^{0.25}}{t^{0.80}}$	1.37	0.52	0.25	0.14	1.61	0.61	0.29	0.17	1.76	0.67	0.32	0.19	1.98	0.76	0.36	0.21	2.32	0.89	0.43	0.24	2.72	1.04	0.50	0.29

It should be noted that Table 17 compares computed values, by formulas based on composite records and duration periods to four days, with observed values from single station records for a duration period of thirty days. For this reason it is thought fair to show the algebraic averages of the deviations.

U. S. Weather Bureau data giving accumulated daily and monthly rainfall depths must be accepted as representing the time interval observations and the calendar month, respectively. Maximum 24-hour and 30-day values would be slightly higher.

The writer's formulas are shown in Table 18, with those published by Charles W. Sherman, M. Am. Soc. C. E., for Boston, Mass.,[16] and by M. F. Wagnitz, M. Am. Soc. C. E., and Louis C. Wilcoxen, Assoc. M. Am. Soc. C. E., for Detroit, Mich.,[17] respectively. They are in practical agreement for the lower frequencies and shorter durations, but the writer believes the comparatively short records, upon which they are based, have given them exponential values which are too high and which will result in intensity rates for the longer durations that will prove to be excessive when their individual records have extended to 100 years.

[16] *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 721.
[17] *Loc. cit.*, December, 1930, Papers and Discussions, p. 2180.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HIGHWAY LOCATION: GENERAL CONSIDERATIONS

Discussion

BY MESSRS. B. P. HARRISON, AND E. P. ARNESON

B. P. HARRISON,[6] M. AM. SOC. C. E. (by letter).[6a]—Too little emphasis is given in this paper to the part that safety should have in the location of highways.

Should not the engineer feel some responsibility for the lives lost due to a faulty location? The roads are crowded with poor drivers and the toll of human life is rapidly increasing. The tendency of the day is for increased speed. Some States have increased the speed limit, while others have even done away with the speed limit entirely. Every one agrees that the loss of life is far too great and that something should be done to lessen the mortality.

Is the engineer doing his part by insisting upon proper locations? In days gone by, with slow traffic, it was probably economical and proper that bridges should be built at right angles to the streams. The writer has seen many examples in which locations have been badly distorted in order to accomplish this purpose. Bridge engineers sometimes fight a location because it will require a bridge to be built crossing the stream on a skew. The time has come when the highway engineer must insist upon the proper location, whether it will require a bridge to be built on a tangent, a curve, or a skew. The hazard is too great to permit the old right-angle crossing with its dangerous curves at or near each end of the bridge. Generally speaking, engineers must build the bridge to fit the location and not make the location to fit a bridge site.

Often there are restrictions, as to sight distance, or curvature that does not require super-elevation. If the sight distance required in a specific case

NOTE.—This paper by Gibb Gilchrist, M. Am. Soc. C. E., was presented at the Joint Meeting of the Highway and Construction Divisions, Dallas, Tex., April 25, 1929, and published in November, 1930, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: February, 1931, by Messrs. C. S. Mullen and H. L. Thackwell.

[6] Associate Highway Engr., U. S. Bureau of Public Roads, Washington, D. C.

[6a] Received by the Secretary, February 24, 1931.

is 500 ft., how seldom does the motorist have any more? The tendency is to do no more than is necessary "to get by". If it should be required that all curves of more than 6° be superelevated, the curves will be made 5° 58′, to avoid superelevating. Why should there be the tendency to do no more than is absolutely necessary to "get by", with any location and plan? Economy has played too great a part in past locations. The engineer responsible for highway locations should spend a little more money, make highways "fool" proof, cut out grade crossings, and thus save many lives. The locating engineer has a wonderful opportunity. Will he take advantage of it?

E. P. ARNESON,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—The author's views are refreshing and stimulating, dealing as they do with the human element in highway location. His conviction, that the people want the highways located for the interest of the many and not for the few, may well be taken by the locating engineers of America as a tenet of faith. All rules of highway location flow as corollaries from that proposition.

The author makes another sententious and challenging statement—that the life of a road is not so much dependent on the type of its surface as on its location. There is an implication here as to the importance of the time element, which the writer would like to discuss.

Referring to the original interest to be served—that of the many as opposed to the few—one knows that this interest may change with the passing of the years. Therefore, the influence of the time element should always be weighed when one is about to fix the position of a proposed highway. The need for the road may be greatly lessened, or indeed it may virtually disappear, in a number of years, by reason of changes in the habits of the people served. The point is, as the author states, that not only does the road wear out, but its very location may have a limited life.

Because of the influence of the time element one may be justified in giving a road an impermanent location, if the ascertained facts warrant. The writer remembers a case in point, where two highways were to be located on general routes which converged on a city with an angle of about 45 degrees. The Northeast Highway, being the more important, was run on a line as nearly direct as the topography permitted. For the East Highway there was the choice of an independent and direct location into the city, or of a combined location by joining the Northeast Highway six miles outside the city. In the latter alternative about $360 000 in first cost could be saved at the expense of some lost distance for the East Highway traffic.

The solution of this problem resulted about as follows: A reasonable estimate of the future traffic on both highways indicated that the joint track would be safe from congestion for twenty years. Moreover, the comparison of the costs and benefits of the independent route for the East Highway indicated that it would take about twenty years for the benefits to pay for the extra investment of $360 000 and its interest. The decision was to combine

[7] Irrig. and Highway Engr. (Walton & Arneson), San Antonio, Tex.
[7a] Received by the Secretary, August 31, 1931.

the two highways, reasoning that it was unwise to build the more direct and independent location of the East Highway, when it would apparently take as long as twenty years to pay for itself, provided the future developed as predicted by the engineer. Nevertheless, the time may come when the East Highway can properly be given an independent and direct entry to the city. Then the road, as theretofore located, will be one more route cast into the limbo of impermanent locations. Permanency of location is a relative matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FLOW OF WATER IN TIDAL CANALS

Discussion

BY RALPH G. DAVIS, M. AM. SOC. C. E.

RALPH G. DAVIS,[40] M. AM. SOC. C. E. (by letter).[40a]—In this paper, the author has expressed regret that sufficient observations were not available for a close comparison between the theory and the observed tides and currents in the Chesapeake and Delaware Canal. Some observations have been made since those to which the author refers and it is the purpose of this discussion to compare the observations with the results computed by the theory.

Colonel Brown's results are not used for the comparison, as it was found that the canal cross-sections at the time (1928) the observations were made were not uniform and, in some places, gave appreciably different working data. Therefore, in order that a close comparison might be made, the theory has been used and extended to fit the conditions at the time of the observations, and new values have been derived. Additional work has been done on the canal since the preliminary observations (May 13, 1927), which affects the tidal regimen materially; so it seems desirable to include a brief description of the canal as it was at the time the later observations were made.

The channel was dredged with a horizontal bottom to a depth of 12 ft. below mean low water, Delaware River datum. It is 150 ft. wide, from the 12-ft. curve in the Delaware River to a point about 500 ft. west of Reedy Point Bridge (see Fig. 1); 90 ft. wide to the lift bridge at Chesapeake City; and 150 ft. wide in Back Creek, to the 12-ft. curve in Elk River. All side slopes were dredged to 1 ft. vertical on 2 ft. horizontal. As the cut was made through low-lying marsh areas on the eastern end, a high divide near the middle, and in the bed of Back Creek, which varies from 350 to 1 400 ft. in width, the different classes of materials excavated resulted in variations in

NOTE.—The paper by Earl I. Brown, M. Am. Soc. C. E., was published in December, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: February, 1931, by R. L. Faris, M. Am. Soc. C. E.; March, 1931, by Messrs. G. T. Rude, Eugene E. Halmos, and W. M. Black; and May, 1931, by H. F. Flynn, M. Am. Soc. C. E.

[40] Engr., Beach Erosion Board, Office of Chf. of Engrs., U. S. Army, Washington, D. C.

[40a] Received by the Secretary, August 19, 1931.

the depth and width of the completed channel. In some places, considerable overcutting was done to provide for the slides and bulging of the bottom.

Subsequent operations included the dredging of the Delaware City Branch Channel, 6 ft. deep at mean low water and 50 ft. wide on the bottom, from the Delaware River, at Delaware City, to the 12-ft. canal, a distance of about $1\frac{3}{4}$ miles. The point of junction of the two channels is about 2 miles west of the outer ends of Reedy Point jetties. The Branch Channel was constructed before the current observations were made and preliminary computations indicated that this "by-pass" has a marked influence on the tides and currents throughout the greater part of the canal. The 150-ft. channel at the western end is a component part of the sea-level canal and is affected, to some extent, by the Delaware River tides propagated through the canal. Therefore, this section has been included in the computation for the canal proper.

For the writer's computations, the canal was divided into four sections, exclusive of the Branch Channel, as shown by Fig. 23. Section 1, about $3\frac{1}{2}$

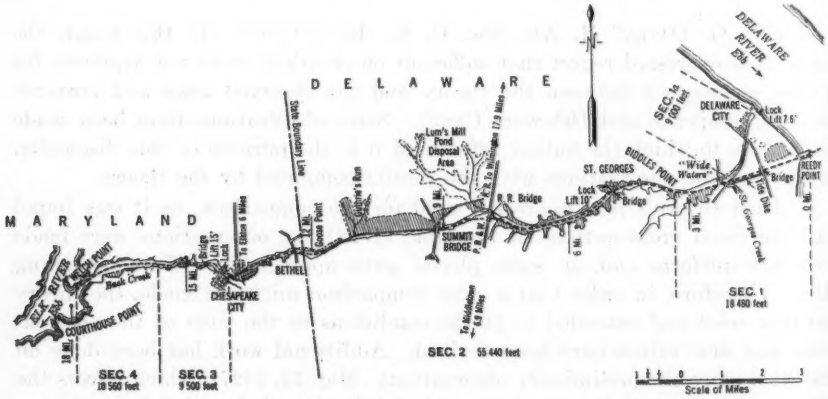


FIG. 23.—CHESAPEAKE AND DELAWARE CANAL, SHOWING COMPUTING SECTIONS

miles in length, includes the 150-ft. channel at the eastern end and the wide waters through the low marshy area. In this section, it was found that the average width at high water is about twice the low-water width, and in the computations, the two widths were handled separately for the positive and negative waves. Section 2 extends from the $3\frac{1}{2}$-mile point through the deep cut section of the canal to Chesapeake City and is about $10\frac{1}{2}$ miles long. Back Creek was divided into two sections, according to the average widths. Section 3 is about $1\frac{7}{8}$ miles long and Section 4, 2 miles in length. The Delaware City Branch Channel Section 1(*a*) has a reduced depth nearly equal to that for the positive wave of Section 1, and a little greater than one-half the reduced depth for the negative wave, which affects the tides in the main 12-ft. canal considerably. The values for the various sections, as used in the computations, are shown in Table 12.

TABLE 12.—DIMENSIONS OF CANAL FOR TIDE AND CURRENT COMPUTATIONS.

Section (see Fig. 23).	1(a)	(1)		(2)	(3)	(4)
Canal Stations (thousand feet):						
From	.....	3+600		22+080	77+520	87+020
To	.....	22+080		77+520	87+020	97+530
Distance from Reedy Point tide gauge to western end of section, in feet	10 500	18 480		73 920	83 420	93 980
Length of sections, in feet	9 000	18 480		55 440	9 500	10 560
		Negative wave.	Positive wave.			
Average cross-sectional areas, in square feet	743	2 032	4 323	2 446	3 705	6 932
Average width, in feet	122	240	461	185	450	1 130
Reduced depth, in feet	6.09	11.22	6.63	13.22	8.23	6.13
Average wetted perimeter, in feet	128	245	468	192	455	1 134

The relation between the results of the computations and the observed tides and currents is shown graphically on Figs. 24 to 28, inclusive. The observed tide curves (solid lines) represent the average for a long series of tide readings at the five stations indicated. The current curves are averages for five full tide cycle observations by current meters held in the center of the channel opposite the tide stations and at the six-tenths depth through the entire tide.

Theoretical values agree fairly well for the intermediate stations, the greatest difference occurring in the velocities at the ends. At Reedy Point and Biddle's Point Stations, at the eastern end of the canal, the observed negative velocity curves are somewhat irregular (see Figs. 24 and 25). These

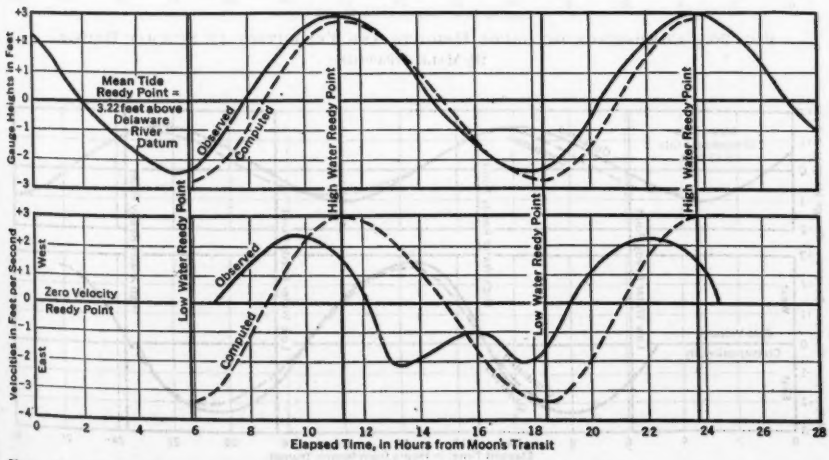


FIG. 24.—COMPARISON OF GAUGE HEIGHTS AND VELOCITIES, OUTER END OF JETTIES, REEDY'S POINT, STATION 0, MILES

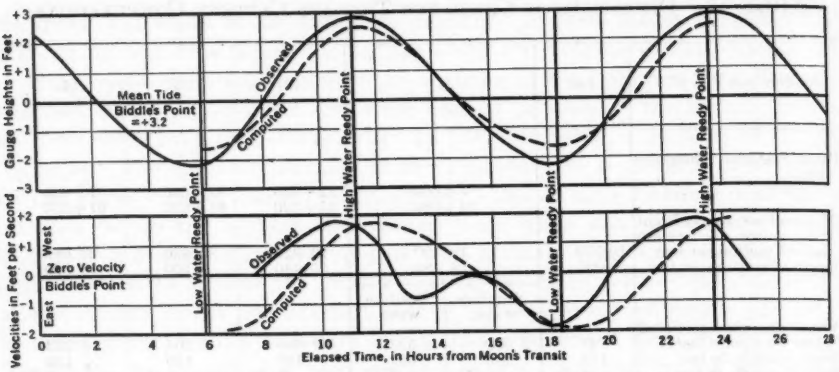


FIG. 25.—COMPARISON OF GAUGE HEIGHTS AND VELOCITIES AT BIDDLES POINT; 3-MILE STATION

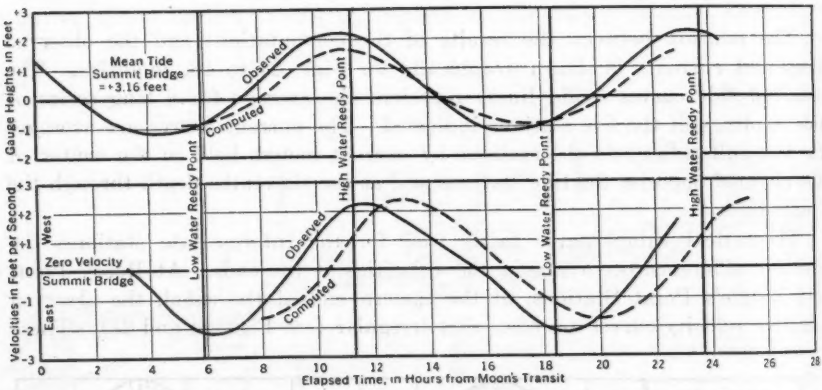


FIG. 26.—COMPARISON OF GAUGE HEIGHTS AND VELOCITIES AT SUMMIT BRIDGE; 9½-MILE STATION

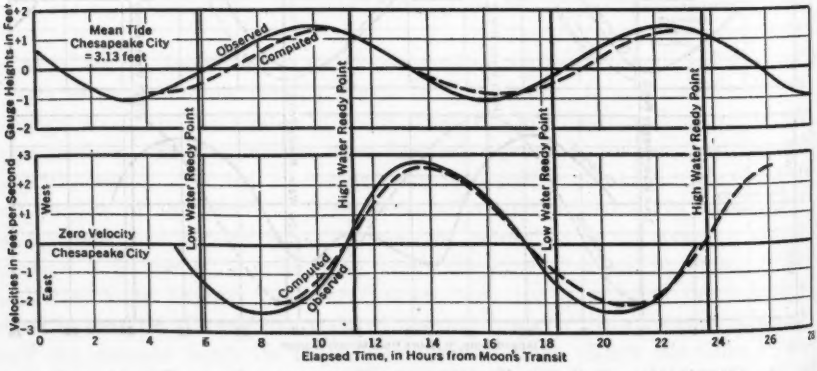


FIG. 27.—COMPARISON OF GAUGE HEIGHTS AND VELOCITIES AT CHESAPEAKE CITY; 14-MILE STATION

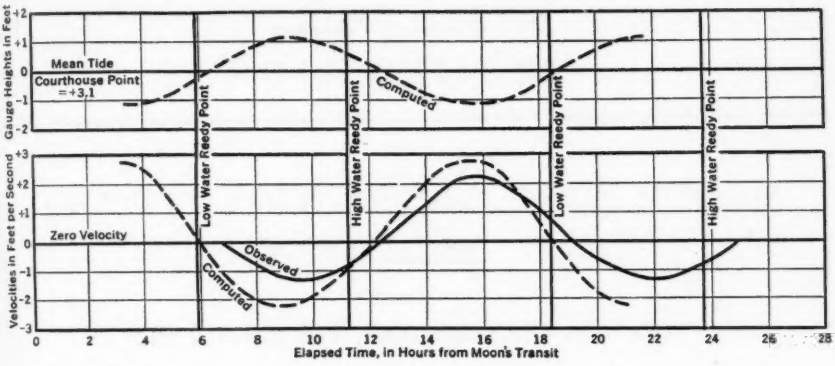
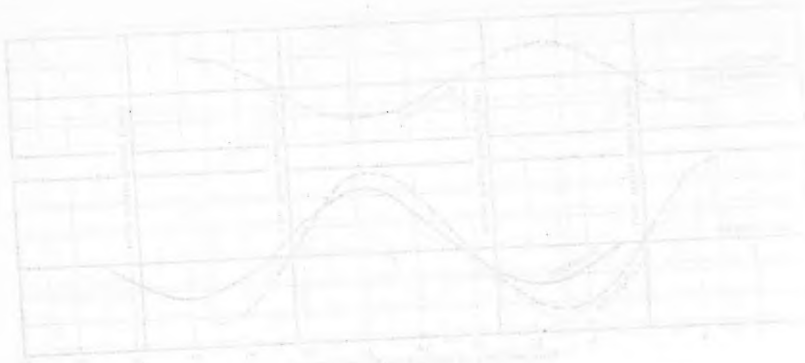


FIG. 28.—COMPARISON OF GAUGE HEIGHTS AND VELOCITIES AT COURTHOUSE POINT (ELK RIVER); 18-MILE STATION

irregularities are apparently due to hydraulic phenomena producing alternate siphoning effects at the junction of the two channels. The divergence at the western end is probably due to the difference in the sectional area of Back Creek gauging station and the average area for Section 4, as used in the computations. The comparison demonstrates that the theory is in close agreement with the observations and that it is a practical method for predetermining the tidal heights and velocities in a tidal canal designed to connect two bodies of water, each having an independent tide of its own.



AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

GRAVITY DAMS ARCHED DOWN STREAM

Discussion

BY MESSRS. B. F. JAKOBSEN, AND A. FLORIS

B. F. JAKOBSEN,[7] M. AM. SOC. C. E. (by letter).[7a]—The experimental investigations reported by the author are interesting and illustrative, and to many engineers, no doubt, more satisfactory than pure theoretical investigations. The experimental investigations are based on the dam as a continuous whole, and the results are just what theory leads one to expect. The writer considered the effect of arching a gravity dam some years ago, treating the dam as a whole, and concluded[8] that,

"Arching in this case, therefore, means that a definite disadvantage, the reduction in stability, is exchanged for a very problematical gain due to arch action."

When a thin slice of dam is considered, as in the customary analysis, it can be readily shown that arching decreases the gravity action.[9] P. Wilhelm Werner, Assoc. M. Am. Soc. C. E., has shown the increase in stability which is obtained when the gravity dam is arched down stream instead of up stream, as is usual.[10] He applies his formulas to the Exchequer Dam and compares the stresses found with those in the dam as built, arched up stream.

When considering the dam as a whole, it is necessary, as the author points out, to bear in mind that shrinkage cracks or planes of weakness may exist, so that the dam, in fact, can not act as a whole. This might be overcome, of course, by constructing proper contraction joints, or by grouting, or by both methods. When a dam rotates on its toe, it is difficult to determine with any degree of precision what the stresses are at the contact between the toe and the foundation. This would depend on the load to be carried by the toe and on the deformations of the concrete and the rock. In the case of the St. Francis Dam there were evidently definite planes of weakness, more or less

NOTE.—The paper by B. J. Lambert, M. Am. Soc. C. E. was published in March, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: In August, 1931, by Messrs. Eugene Kalman, William P. Creager, and T. L. E. Haug.

[7] Cons. Engr., Los Angeles, Calif.

[7a] Received by the Secretary, April 20, 1931.

[8] "Arching Not Effective in Slightly Arched Gravity Dams," *Engineering News-Record*, June 12, 1924, p. 1020.

[9] "Notes on Arched Gravity Dams," by B. F. Jakobsen, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 766.

[10] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 784.

vertical and radial, so that the dam did not act as a whole; and, furthermore, there appears to have been spawling at the toe of the maximum section. On the whole the safer design will generally result when thin sections of a dam are considered. The only exception to this may be where the height of the dam changes rapidly, so that the longer cantilevers lean upon the shorter, as the writer has discussed elsewhere.[11] Small dams might conceivably be considered as a whole, especially if the abutments are not too steep.

The writer does not look with favor on Professor Lambert's suggestion of anchoring the dam by steel tension members. It might be useful and acceptable where the rock is especially good and free from fractures and for a more or less temporary structure.

The tests are a welcome addition to engineering literature, and it is gratifying to see that they agree so well with theoretical deductions.

A. FLORIS,[12] ESQ. (by letter).[12a]—In this paper the author attempts to investigate the statical behavior of gravity dams in relation to the topography of the site. This method seems to be quite logical. However, it requires that the structure be analyzed as a whole instead of by the customary practice of considering vertical slices only. This last simplification is made for the purpose of facilitating the stress analysis of gravity dams; but, nevertheless, an exact stress determination with regard to the foundation conditions of the dam is not yet available.

By proposing to consider the entire dam as acting in one piece, the author introduces additional difficulties which render even an approximate analysis of gravity dams very difficult. He restricts himself to the study of overturning movements of concrete dam models with special reference to their site, but his observations are not expressed in a precise mathematical form. In the paper, general methods are not given by which the limiting cases can be ascertained and thus the place at which the dam begins to overturn, or at which failure begins to occur due to excessive stresses, cannot be determined.

In a rather unsatisfactory manner Professor Lambert considers equilibrium conditions only, as if he was dealing with perfectly unelastic bodies. How great are the stresses and how are they distributed under the influence of the weight of the dam and the water pressure before overturning? These questions are not answered satisfactorily. Those who advocate innovations in the design and analysis of structures, must follow their ideas to the end and solve the problem completely. The study of the stability of dams by the aid of relatively large models, as suggested by Professor Lambert, are of no value, if an adequate theory of the relation between dam and site is lacking.

A fair comparison between dams curved up stream and those curved down stream can be made, if the structure is considered purely as a gravity dam. Otherwise, the dam curved down stream is decidedly at a disadvantage over the one curved up stream. As dams are made from concrete, a material of low tensile strength, it follows that the ones curved down stream (which are cylinders subjected mostly to tension) are inferior to those curved up

[11] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 835.

[12] Civ. Engr., Los Angeles, Calif.

[12a] Received by the Secretary June 27, 1931.

stream, stressed mainly by compression. The latter are only arches and, therefore, the author's usage of the expression, "gravity dams arched down stream," is somewhat misleading.

The standing section of the St. Francis Dam does not bear testimony in favor of either the down-stream or the up-stream forms. It indicates merely that in dams with a great radius of curvature in plan, gravity action only can be relied upon. Apparently, in the central part of the St. Francis Dam, the foundation conditions were better than in the two wings. This fact was stressed in the various reports on the failure of this dam.[13]

The suggested use of chains for the purpose of preventing gravity dams from overturning and sliding, is quite unusual and, it may be said, out of the question from purely practical considerations. Gravity dams with their great quantity of concrete, implying strength and durability, are incompatible with thin steel chains having a restricted life of usefulness. From the theoretical point of view these chains complicate the stress analysis of gravity dams, which is a fact overlooked by Professor Lambert.

As far as the writer knows, gravity dams curved down stream were first suggested[14] by P. Wilhelm Werner, Assoc. M. Am. Soc. C. E.

[13] "Essential Facts Concerning the Failure of the St. Francis Dam," Report of Committee of Board of Direction, *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2147.

[14] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 784.

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DISCUSSIONS

FINANCING STREET AND HIGHWAY IMPROVEMENTS

Discussion

BY MESSRS. A. P. GREENSFELDER, AND C. R. THOMAS

A. P. GREENSFELDER,[6] M. AM. SOC. C. E. (by letter).[6a]—An analysis of the broad principles involved in financing highway improvements has been ably presented in this paper.

There would seem to be no worthy reason for diverting for other than highway use the funds levied on motor vehicles and fuel. In spite of the equity in the case, a number of States are using gasoline tax money for a variety of other purposes. To date, however, motorists have been found willing to support these taxes because of their desire to secure ample highway facilities. If American legislators break faith and use such gasoline funds derived from motor vehicles for other than motor highways, there is likely to be "back fire" which they will not enjoy. America has needed, and still needs, thousands of miles of major highways. The user of these highways should be given every consideration because he is paying the major part of the bill.

In determining the road user's share of costs, it would seem essential to consider the separation of highway bond funds into road-bed bonds and paving bonds. The road-bed bonds should include costs of right of way, drainage, and grading. These costs might be allocated to the various political subdivisions of the Government, and might be issued for a term of thirty to fifty years. Serial bonds should be issued only for the estimated life of the particular pavement and then might be allocated logically to the users, including amortization, interest, and maintenance of the highway.

Just as it is logical to tax motor vehicles for highways, so it would seem only equitable that fuel tax from inland vessels be allocated to the cost of providing navigable inland waterways, and the fuel tax from aircraft should be allocated to airport and airway lighting.

NOTE.—The paper by R. W. Crum, M. Am. Soc. C. E., was published in August, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[6] Pres., Fruin-Colnon Contr. Co.; Pres., Associated Gen. Contrs. of America, St. Louis, Mo.

[6a] Received by the Secretary August 28, 1931.

Relative to primary roads, it would seem that the community benefits derivable from private trucking and buses should be weighed against the very probable damage and heavier pavement costs which these vehicles will presently cause. Railways are required to have private rights of way and yet State and interstate truck and bus traffic is permitted to block the highways without regard to other vehicles. Perhaps long hauls by rail upon private rights of way and local distribution from pivotal centers by trucks on public highways will be found to be the economic answer. The writer agrees with Mr. Crum that there is need for co-operation between all transportation groups. There is danger, however, that highways may be damaged by fast heavy truckage of low-class freight more than the gain in freight charges or the value of time saved in delivery.

There is little doubt but that automobile transportation of individuals is extremely expensive per capita per mile. That the saturation point per capita has been temporarily reached is evidenced from the 1931 automobile sales. Whether individual passenger automobiles per mile per capita will increase or decrease is debatable.

In considering sources of funds, it is interesting to recall that in 1880 when local highways were desired, the benefited parties were frequently required to pay direct toll. While toll highways have been practically eliminated, toll bridges are still being constructed for the same obvious reason, namely, that the toll charge is more than justified in the time and mileage saved. Tolls are merely direct taxes paid by the benefited users, whereas the gasoline fund is an indirect tax.

Federal aid may have been more justifiable in former times than at present. It is easier to educate from the top down than from the bottom up. Federal aid served as a bonus to stimulate State and county highway construction. Just how long and how much the National Government should continue to contribute would seemingly be a function of interstate vehicular traffic.

It seems only equitable that both primary and secondary roads should be paved through a city or village as a continuous highway. That this was not done originally was due to the fact that county legislators overwhelmingly predominated the representatives of the urban centers in any plan. In justice to the city taxpayers who collectively contribute a very large sum for highway construction, the through highway should be continuous. It seems equally vital that the expenditure of funds for through highways in cities should be under the control of the State Highway Department, the same as it is throughout the counties. If the State Highway Department can design, contract, maintain, and pay for the highways in the counties of the State, it would seem perfectly reasonable that they should do the same thing through the cities of such counties, since both are political subdivisions of the same State.

There is another type of highway which Mr. Crum does not mention, namely, those that go to and through various State Parks. American citizens to-day demand outdoor recreational facilities and are increasing the number

of National, State, and County Parks, which, in turn, increasingly require highways to reach them. In a bond issue of $75 000 000, in Missouri, several million dollars were thus allocated in 1928.

In connection with third-class roads the author questions just how much the land owner should contribute. It seems fundamental that the least the farmer can furnish is a fair amount of right of way for the highway which will undoubtedly benefit his farm. Perhaps a tax for advertising signs per square foot per 1 000 passers-by per annum would furnish a considerable sum for highway improvements, since without the highway the signs are worthless. Signs are distasteful to most motorists and to tax them thus would minimize their area.

In discussing city streets, the author very appropriately presents the problem of parking space. It would be interesting if the cities would consider providing parking space on unpaved city blocks rather than on the paved street itself. Perhaps it would decrease traffic congestion and, at the same time, be considerably less expensive per car parked. In the down-town section the city might make inducements to private capital for the construction of multi-story garages.

The writer concurs with the author that continuous research along these lines is imperative. The traffic surveys made in the City of Cleveland, Ohio, and in the State of Connecticut during the years 1928, 1929, and 1930, are splendid examples of what can be done. City, county, and regional planning must be based on adequate information.

C. R. THOMAS,[7] M. AM. SOC. C. E. (by letter).[7a]—The largest single factor in the stabilization of highway programs is the relative continuity of the income from motor vehicle taxes used for highway purposes. The marked diversion of such funds to purposes other than roads and streets would be a great blow to stabilization of income that permits budgeting of funds several years in advance, and to the amount of road building now (1931) in progress. As Mr. Crum states, the problem of allocation of the motor vehicle taxes among various classes of roads and streets or governmental units, which is controversial, is overshadowed by the greater problem of stopping diversion to purposes other than highways.

The principal reasons advanced for the diversion of the motor vehicle taxes to other purposes than roads are: (1) Other State activities need money more than roads; (2) the road system is well advanced toward completion, and the funds raised by motor vehicle taxes should go to other purposes; (3) motor vehicle taxes belong to the general funds of the State and can be used as the State legislatures desire. These reasons are advanced with the object of influencing legislatures so that appropriations may be obtained for many purposes foreign to roads and streets. Back of these reasons may be the desire of property owners to use motor vehicle taxes as a substitute for levies on real estate.

[7] Engr.-Executive, Am. Road Builders' Assoc., Washington, D. C.

[7a] Received by the Secretary, September 3, 1931.

The original purpose of motor vehicle taxes, which include both gasoline and license taxes, was to provide roads for motor vehicles. Taxes are assessed for schools and other special purposes and diversion of such taxes is not generally considered. The justice of the motor vehicle taxes (which are really "use" charges in lieu of tolls), like telephone charges, has been proved by the apparent popularity of the tax and the lack of opposition to repeated increases. However, some opposition to the motor vehicle taxes has developed since diversions have taken place in seventeen States, so that the motor vehicle drivers no longer receive full benefits. The growth of the idea for diversion has followed the increase in returns which amounted to $850 388 000 in 1930, and the demands of other State interests for a part of this large sum of money. Diversion to purposes other than roads amounted in 1930 to 3% of the gasoline tax income. Practically all the diversion is from the gasoline tax.

The reason advanced that other State activities need the money more than roads opens up the whole field of discussion of the value of highways, both tangible and intangible, as compared with the public value of other State activities. The subject is too broad for discussion here. It is sufficient to state that the savings in transportation costs alone to the public more than pays the cost and sinking fund for replacement of heavy traffic highways. No other State activity appears to be on such a sound financial basis. The danger of changing from an investment that is returning such tangible dividends is at once apparent.

While more roads are being paved in 1931 than a few years previously, the highway system is by no means approaching completion for several reasons. There are 3 000 000 000 miles of public roads and only 700 000 miles have been surfaced. New developments in vehicles, such as the change in the few years preceding 1931 from low to high road speeds, bring new burdens on the roads that call for complete revision of highway-design practice. The rapid increase in the number of motor vehicles has led to congestion on the highways with resulting costly delays and accidents. Highway facilities are far behind the motor vehicle facilities both as to rate of production and mileage of improved roads. It has been stated that in Cook County, Illinois, if all the vehicles in the county were on the improved roads at one time, they would be two deep.

The idea that motor vehicle taxes belong to the general fund of a State is perhaps sound in some States, due to constitutional provisions, but the use of such funds for purposes other than roads and streets is undoubtedly a violation of the intent of the motor vehicle taxes as originally imposed, and is contrary to the public understanding of the purpose of the taxes. It is certain that opposition to motor vehicle taxes will develop in almost direct ratio to the amount of the diversion to illegitimate purposes.